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1 December, 1953

Mr Arthur Floyd, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Road Paper No. 43

"The Design and Construction of a High-Speed Test Track for Motor Vehicles"

by

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SYNOPSIS

The Paper describes the conversion of the flat perimeter track of a disused airfield into a modern high-speed vehicle-testing circuit for the Motor Industry Research Association, with steeply superelevated bends, permitting average speeds of up to 125 miles per hour to be maintained. With the exception of the old Brooklands race track, the work is believed to be without precedent in Great Britain.

Two sides of the existing triangular circuit are incorporated, and a new third leg about $\frac{5}{8}$ mile long has been added. With the three new bends, the total length of the circuit is about $2\frac{3}{4}$ miles. The new bends consist of 9-inch concrete paving, having a maximum superelevation angle of 1 in $1\frac{1}{2}$, cast in situ without after-treatment of surface, and founded on embankments of clay excavated from borrow-pits on the site. The new leg to the triangle has an asphaltic surface and is founded on a flexible base.

Close tolerances were called for, including a maximum divergence of $\frac{1}{8}$ inch from a 10-foot straight-edge. A first-class surface has been achieved, and the design of transition and circular curves allows the bends to be negotiated safely at 100 miles per hour with reliance on side friction to the extent of only one-sixth of the weight of the vehicle.

Brief descriptions are given of similar tracks in America and France. The considerations leading to the design finally adopted are discussed. The details of design and construction of the clay embankments and concrete pavement are described, with particular emphasis on the problems of achieving the desired general surface shape and accuracy of finish on the concrete placed at high slopes.

Introduction

In 1948 the Motor Industry Research Association took over Lindley airfield near Nuneaton for use as a vehicle proving ground. Various testing facilities were installed on and about the three main runways and perimeter tracks, including timing straights, water splashes, a dust tunnel, and stretches of corrugated track and Belgian *pavé*.

The 50-foot-wide concrete perimeter track served as a circuit for high-speed work. But the speed check imposed by the sharp radius and absence of superelevation at the corners of this triangular circuit was a severe drawback, reducing the average lap speed of a vehicle to considerably below its maximum speed on the straight, and causing excessive tire wear and undue strain on drivers. The conversion of this circuit into a modern high-speed track with steeply superelevated bends forms the subject of this Paper.

Alternative Schemes

The lay-outs considered ranged from a circuit allowing a maximum "balanced" speed of 60 miles per hour (the speed at which the resultant action of the weight and centrifugal force is normal to the track surface) and envisaging later, extension, to 100 miles per hour, to a circuit designed for a balanced speed of 100 miles per hour. Some schemes incorporated all three sides of the existing circuit; others, including the one eventually chosen, used the two straight sides only and replaced the third "dented" side with a new straight leg. Study of these schemes and their estimated costs showed:

(1) that the eventual cost of widening a 60-mile-per-hour track to suit 100 miles per hour would not be economically sound; and
(2) that a track built for 100 miles per hour "balanced" speed initially would be very costly and would shorten the circuit.

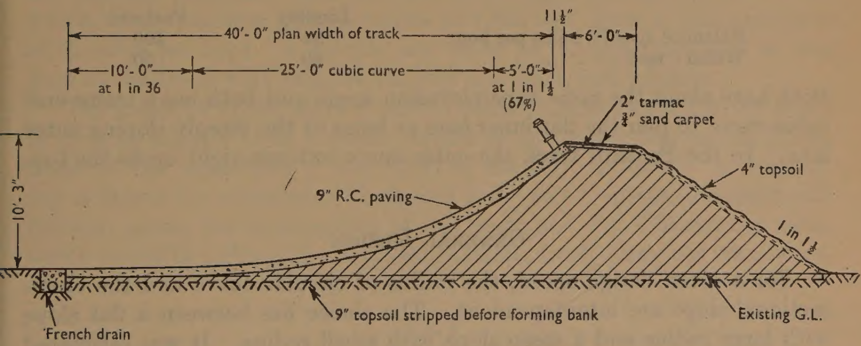
These considerations led to the adoption of a lay-out which was judged to be the best compromise in terms of cost and existing site conditions. This allowed a balanced speed of 84 miles per hour and would permit 100 miles per hour with reliance on side friction to the extent of one-sixth of the normal reaction.

General Description of New Track

The new circuit is triangular. Two of its sides are existing legs of the perimeter track; the other is new and slightly curved to lie within the site boundaries and to avoid certain buildings. The new corners comprise six virtually identical transitions and three circular curves of the same radius, but of different lengths. The maximum superelevation angle is 1 in $1\frac{1}{2}$ ($33^\circ\ 42'$). The total length of the new circuit is $2\frac{3}{4}$ miles, of which about

1¼ mile is new concrete, ⅝ mile is new flexible pavement, and the remainder is existing track. Tenders were invited in November 1951 for the construction of these new works, including drainage and an access subway under the track. Work began in March 1952 and was completed in May 1953. The total cost was £186,000. At one time, seven scrapers and seven smooth-wheeled rollers were employed on the earthworks. The banked bends required 60,000 cubic yards of earthworks and 30,000 square yards of 9-inch concrete paving. The new slightly curved leg (known as the " North-west Straight ") has an area of 15,000 square yards and a thickness of 15 inches of flexible pavement consisting of ashes, limestone base course, two-coat tarmac, and asphalt surfacing. Fig. 1, Plate 1. shows the lay-out of the ground and the arrangement of the new circuit,

Fig. 2



Cross-Section through Circular Curve

Fig. 2 is a cross-section through a circular curve ; views of one of the completed corners are shown in *Figs 14* and *15* (facing p. 197).

Similar Tracks

At the outset, reference was made to information about similar circuits elsewhere. The two circuits of principal interest were those in America—one constructed by Packard, the other by General Motors. These are constructionally similar and it appeared that the track at Lindley should follow the same lines, namely, that the banked bends should consist of in-situ concrete paving founded on embankments built of local material. A soil survey showed that the clay underlying the site would be quite suitable for this purpose.

The French track at Montlhéry consists of precast slabs resting on a framework of concrete beams and columns, but bearing in mind the suitability of the local soil, it was evident that this form of construction would

be considerably more expensive at Lindley. The plan shape and a cross-section through a circular curve are shown in Figs 3, Plate 2, for both the Packard and the Montlhéry tracks.

The most recent, with the exception of Lindley, is the Packard track. This was built in 1926, but it was damaged during the war by tracked vehicles, and complete resurfacing was carried out in 1946. The new surface is a 6-inch thickness of concrete paving on top of the original 6-inch slab. There thus existed an excellent rigid base for the setting up of transverse forms along the transitions and for bearing the weight of a specially designed compacting machine for the circular curves. Since the total length of circular curve on this track was more than three times the corresponding length at Lindley, it was worth building this machine, which was not adaptable for use on the transitions.

The main differences between the Lindley and Packard tracks are as follows :—

	Lindley	Packard
Balanced speed : miles per hour . . .	84	100
Width : feet	40	50

Both have about the same superelevation angle and both use a transverse cubic curve to join the flat inner lane or lanes to the steeply sloping outer lane. In the Packard track the cubic curve extends right up to the top.

General Design

For a given design speed, the radius of circular curve and the cross-sectional slope are interdependent. The choice lies between a flat slope with large radius and a steep slope with small radius. It was estimated that the extra cost of banking for high angles would be more than offset by the saving in length of curve, provided that the concrete could be placed without top shuttering. It appeared that 1 in $1\frac{1}{2}$ was about the maximum slope at which the surface of the concrete could be finished satisfactorily ; this limit was therefore adopted. The corresponding radius for a balanced speed of 84 miles per hour is 710 feet.

Highway transition lengths are often based on a rate of gain of centripetal acceleration of 1·5 feet per second per second per second, although at least one American authority allows 3 feet per second per second per second. It was felt that highway practice would be conservative for a track of this nature and, since transition lengths would have a marked bearing on the total cost, it was decided to adopt about 4 feet per second per second per second at 84 miles per hour (6·5 at 100) giving transition lengths of 685 feet.

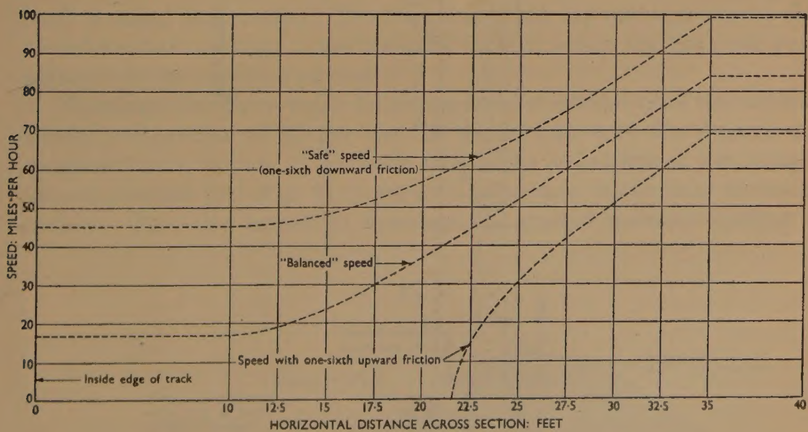
Thus, superelevated curves of 710 feet radius are introduced at the three corners and joined to the straights by transitions, each 685 feet long. The form of the transition is a spiral (radius inversely proportional to distance from tangent point) ; this, with uniform application of super-

elevation along all lines parallel to the centre-line, ensures that the path followed by a vehicle in its correct speed lane and moving freely will be parallel to the centre-line.

The cross-section of the track (see *Fig. 2*) at full superelevation is cubic except for a 10-foot-wide lane at the inner edge sloping at 1 in 36, and a 5-foot width in plan at the outer edge sloping at 1 in $1\frac{1}{2}$. The straight cross-slope at the inside edge affords passing room for slower-moving traffic, whilst the straight outer section gives a fast vehicle the full benefit of the superelevation. The cubic curve between these two straights ensures equal lane-widths for equal speed-ranges. Along the transitions the height of the surface along any line parallel to the centre-line changes at a uniform rate. Thus, any cross-section along the transition is also cubic with straight sections on the outside and inside. At the beginning of each transition, the cross-section has a straight fall of 1 in 36 for drainage purposes. The North-west Straight has the same cross-fall which, in conjunction with its radius of 9,325 feet, gives a balanced speed of 62 miles per hour. A vehicle can travel at 165 miles per hour on this part of the track with reliance on only one-sixth side friction.

The cross-fall at the start of each transition causes a car moving freely to diverge slightly from a track parallel to the centre-line on the flatter parts of the transitions; this effect is scarcely noticeable in practice. *Fig. 4* shows the variations of balanced and "safe" speeds (one-sixth side friction) across the cross-section of the track, and *Fig. 5* shows the paths which would be followed by vehicles travelling freely along the transitions.

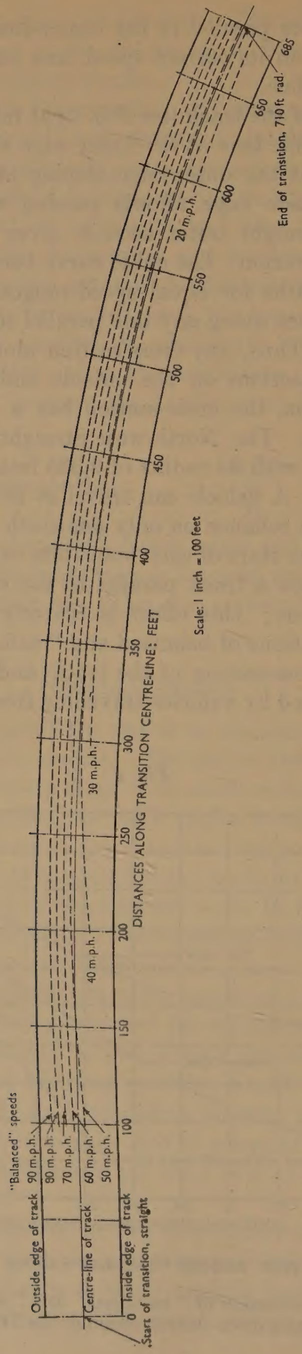
Fig. 4



SPEED VARIATION ACROSS CIRCULAR-CURVE CROSS-SECTION

(This diagram shows the variation of "balanced" and "safe" speeds and speed with $\frac{1}{6}$ upward friction across a cross-section through the 710′-radius circular curve.)

Fig. 5



"Balanced" Speed Variation along Transition

Vertical curves are introduced at changes of grade ; these are designed to limit acceleration normal to the track surface to 4 feet per second per second for upward changes of grade and 2 feet per second per second for downward changes. There are no transitions into these curves. The curves are calculated for the inner and outer edges of the track at the corresponding speeds and normal cross-sections are " hung " from the curves so derived, thus providing a smooth variation of vertical curvature across the width of the track.

Construction Tests

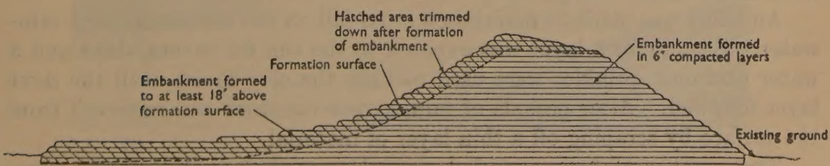
The potential difficulties of placing and finishing concrete at steep slopes were realized from the start ; and a sum of £2,000 was provided in the contract to allow for construction trials to be carried out prior to the main works. A short section of one of the main embankments was formed in advance of the remainder of the earthworks and, on the back slope of this, various methods of trimming to formation were tried and several experimental bays of concrete put down. The concrete mixes eventually used were settled during these tests. A preliminary construction procedure was developed which needed some modification later (see the Section entitled Concrete Paving, Concreting on p. 199).

Earthworks

The embankment has a top width of 6 feet outside the paving slab, sloping away at 1 in 20, and a back batter of 1 in $1\frac{1}{2}$ (see *Fig. 2*). The top width is covered with tarmacadam and sand carpet and the back of the bank is soiled and seeded. The intention of the top seal is to ensure that no substantial variation of the moisture content within the bank can occur adjacent to the concrete paving.

To ensure that there would be no appreciable settlement of the embankments, compaction to 95 per cent of the zero-air-void dry density was specified and achieved. The inside face of the embankment was initially formed oversize (see *Fig. 6*) to ensure compaction at the final formation surface. This also helped to maintain the correct moisture content at

Fig. 6



Embankment Construction

formation during the interval between building the embankment and placing the concrete.

Most of the clay in the borrow-pit areas was found to have a moisture content at or near the plastic limit and proved excellent for compaction. It was specified that all clay used in embankments should have a moisture content within $\pm$ 2 per cent of its plastic limit, the intention being that the embankment would thereby be placed generally at its equilibrium moisture content. About 13 per cent of all material excavated from the borrow-pits had to be rejected, most of this because it was too dry for easy compaction.

Other properties required of the clay were that the liquid limit should not exceed 50 per cent and the maximum dry density obtainable by the Proctor test should be at least 110 lb. per cubic foot. Tests of liquid and plastic limits and Proctor tests were done for every 1,000 cubic yards of material during excavation from the borrow-pits.

Tractor-drawn scrapers placed the embankment in 9-inch loose layers, that is, about 6 inches of compacted thickness, and samples were taken from every layer, their dry densities and moisture contents were obtained, and the degree of compaction was computed. One sample was taken from each 500 square yards of each layer.

To build the embankment to the required profile without unnecessary waste of material called for great care in setting-out and supervision. Profiles, set up behind the embankments, were graduated to show the number of layers required. No profiles were used for the inside of the bank, but pegs driven into each completed layer marked the limit of the next layer. *Fig. 7* shows a completed section of embankment viewed from inside.

Fig. 8 shows a general view of earthwork in progress at the north corner of the track. The plant used consisted generally of D7 or D8 tractors with 9-cubic-yard-capacity scrapers; one 10- or 12-ton smooth-wheeled roller per scraper was needed to enable compaction to keep pace with laying. The greatest number of scraper-roller units employed was seven and the 60,000 cubic yards of embankment were mostly completed during the months of July, August, and September, 1952, the average round trip per load being 300 yards.

The clay, being rather lumpy, was deposited about 1 foot deep; this was then reduced to 9 inches by a second pass of the scraper. The rollers then made eight to ten passes on each layer.

An effort was made to maintain a cross-fall on the surface to shed rain-water. Sometimes a layer was exposed to the sun for several days and a water cart and sprinkler were used to keep the clay moist until the next layer followed. After periods of rain, excess moisture was removed from the surface by scraping off a thin layer of material.

The number of tests specified proved somewhat excessive and some relaxation was allowed towards the end of the work. It was compara-

Fig. 7

Completed Embankment showing Formation Face Before Trimming

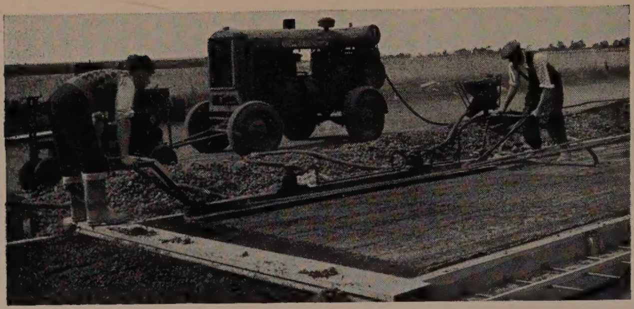
Fig. 8

General View of Embankment Construction

Fig. 9

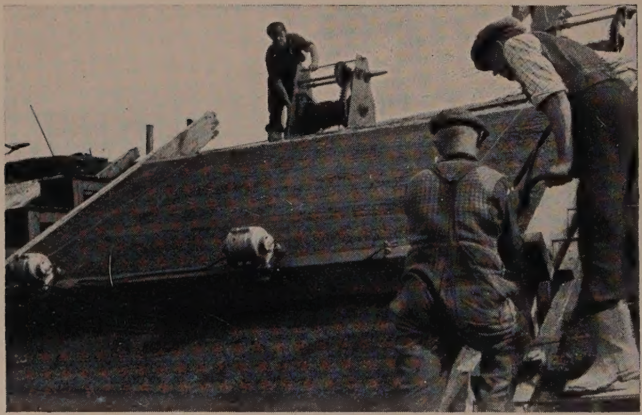
North-West Straight. Laying Single-Size Limestone Base by Spreader Box

Fig. 10



Inner 10-foot Lane. Compacting Bottom Layer

Fig. 11



Vibration of Top Layer on Test Bank

Fig. 12

Compaction of Bottom Layer in Transverse Bay. Hand Finishing of Top Layer in Background

Fig. 13

General View Showing Setting of Forms and Concreting

Fig. 14

Completed South-East Corner

Fig. 15

Car Travelling at about 80 miles per hour on South-East Corner Transition. South-West Corner in Background

tively easy to achieve the 95-per-cent degree of compaction required so long as the moisture content of the clay was at or near the plastic limit. In cases where the material was too dry or perhaps contained sand or silt, the degree of compaction fell to 80–90 per cent. Such material was easily identifiable and was automatically rejected by the contractor. In many cases the results of dry-density tests were not available until the layer to which they referred had been covered, and inspection of the layers after rolling provided a sufficient running check on compaction which the test results usually confirmed.

In periods of dry weather, all possible overtime was worked on the embankments, because the condition of the clay was very sensitive to anything more than a light shower. The first items of plant to be affected were always the smooth-wheeled rollers, which quickly bogged down. From this point of view it might have been better to use pneumatic-tired rollers. Road Research Technical Paper No. 17 [1] mentions that an advantage of these rollers on clay soils is that they can be used at relatively high moisture content.

Fortunately, the clay dried quickly under favourable conditions and it was rarely necessary to wait long following heavy rain before recommencing work.

Trimming of Formation Face

None of the methods tried on the test length proved entirely successful. On the embankments, the trimming operation commenced naturally at the flat ends of the transitions, where the operation was conveniently accomplished by scrapers. The use of large scrapers for rough trimming, followed by small ones, was pursued throughout the transitions and, with practice and skilled driving, this proved satisfactory, even on the 1-in-1½ slopes of the circular curves. Final trimming of high spots was done by hand, using mattocks, and low spots were filled in with weak concrete.

A dry-density and moisture-content test was made on a sample taken from each 100 square yards of trimmed formation to check that the specified degree of compaction had been attained.

Immediately after trimming, a dressing of cold emulsion containing 55 per cent of bitumen was applied and blinded with stone chippings. This was to protect the clay from the effects of rain during the interval before the placing of concrete on it, and proved very successful on the steeper slopes. Some penetration did occur on the flatter areas during the winter weather, but, even in the worst cases, significant alteration of moisture content was limited to a depth of 3 inches.

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[1] The references are given on p. 205.

Concrete Paving

Design

The pavement was required to have a first-class running surface and to support wheel loads of 8,500 lb. at the inner edge, falling to 2,000 lb. at the outer edge (including the effects of centrifugal force). The traffic frequency expected was about 300 vehicle passes a day in any one lane.

The paving slab is 9 inches thick throughout and rests directly on the bituminous seal covering the compacted clay subgrade. The thickness was decided after a study of various design methods,[2] the final choice being largely a matter of judgement based on the values obtained. Compaction and finishing by vibration were specified.

A granular base course, very often used on clay subgrades, was omitted. It would have been impracticable in any case to provide one under the steeply sloping parts of the paving; and in view of the minimum cross-fall of 1 in 36, and the presence of dowels and tie-bars in the slab joints, it was decided to omit it also below the flatter portions. Strips of damp-course felt were provided below all joints as an additional safeguard.

A single layer of mesh reinforcement weighing about 8 lb. per square yard is provided everywhere 3 inches below the surface and to within 3 inches of all joints and edges. Reinforcement near the top of the slab gives good protection against surface cracking and this was felt to outweigh the limited structural advantage of placing it near the bottom.

The pavement was designed for construction in alternate transverse bays, the joints being radial and nowhere more than 14 feet apart. Even at this spacing, vibrating screeds proved to be somewhat unwieldy on the steep slopes. To form the curved surface by a series of straight facets would have meant a sudden change of grade of about 1 per cent at each transverse joint on the steep slopes. This was overcome by using downward-cambered screeds.

It was decided to confine the transverse bay construction to the outer 30-foot width of track and to construct the inner 10-foot lane in longitudinal bays. This was to be done ahead of the transverse bays to provide a base against which to level the transverse templet forms and on which to place construction plant.

Those parts of the track constructed in the winter are provided with transverse dowelled expansion joints at approximately 50-foot (four-bay) centres. The joints are filled with a normal joint filler and sealed with a rubber/bitumen compound. All construction joints are fully tied. One corner of the track was constructed in warm weather; here, two transverse joints out of three allow for contraction only, having dowel bars but no joint filler. This results in considerable movement at the expansion joints and the adhesion of the sealing compound is not sufficient. Having tried this arrangement the Authors now consider that 50-foot expansion joint spacing everywhere would have been better.

It was specified that the transverse templet forms should be fabricated to within $\pm \frac{1}{16}$ inch of the design profile, and that they should be set up so that any point on any one form should be within $\pm \frac{1}{8}$ inch of its required level relative to the two adjacent forms. A local tolerance of $\frac{1}{8}$ inch under a 10-foot straight edge was also specified.

Setting of Forms

The setting of transverse templet forms to meet the above tolerances was an operation requiring great care and constant checking. The forms were made of 9-inch-by-3-inch timber in lengths not exceeding 15 feet. They had to be set on the clay formation and precautions taken against settlement during placing and finishing the concrete.

Setting of the forms along the transitions was done by levelling each form at its joints after careful positioning in a radial direction. Finally, the vertical intercept between a string line and the form was checked against the calculated value at 5-foot intervals across the width of the track, the string line being stretched between the two neighbouring forms. It was found just possible to meet the $\pm \frac{1}{8}$ inch tolerance. This check was repeated immediately prior to concreting, and after packing weak concrete between the form and the underlying felt strip. A general view of forms set up ready for concreting is shown in *Fig. 13* (between pp. 196 and 197). One set of forms was used for the six transition curves and the circular-curve forms also had about six uses.

On the circular curves every fourth form was accurately located by instrument and tape, and the intermediate three set to calculated intercepts from a wire stretched over them, thus minimizing instrument work.

Concreting

A central weigh-batching plant with two $\frac{1}{2}$-cubic-yard mixers was set up. Daily aggregate moisture contents were taken and the workability of the mixes checked by a compacting-factor apparatus.[3] The aggregates available could be combined to give a grading similar to No. 1 ($\frac{3}{4}$ inch) in Road Note No. 4.[4] Weigh-batching with frequent compacting-factor tests gave a very uniform concrete. Test cubes during some weeks gave an 80-per-cent ratio of minimum to average strengths. It should be stressed that the main object of weigh-batching and close control here was to ensure that a suitable mix for placing on the high slopes, once found, would be produced consistently.

All concrete was placed in two layers. A 7 : 1 mix with a compacting factor of 0·82 (water/cement ratio of about 0·50) was used for both layers of the inner 10-foot lane, both layers of the less steeply sloping transverse bays, and the bottom layer only of the more steeply sloping bays. In the latter, the top layer consisted of a 6 : 1 mix at a compacting factor of 0·85 (water/cement ratio of about 0·45). The higher compacting factor for the concrete placed on the steep slopes may be thought

anomalous. This mix was, however, devised after a number of trials with leaner mixes and had substantially the same ratio of mortar to stone as the 7 : 1 mix, the mortar being enriched by the replacement of some of the sand and water by cement. This gave the mix the cohesive property necessary to prevent excessive flow under vibration.

The specification called for compaction and finishing entirely by vibration except on the very steeply sloping parts where it was foreseen that hand finishing might prove necessary. At the first corner, the inner 10-foot lane was accordingly constructed in this way. In both layers the concrete was spread as evenly as practicable and without excessive trampling to a predetermined surcharge. *Fig. 10* (between pp. 196 and 197) shows the surcharged bottom layer before and during compaction.

It proved impossible, however, to produce a surface meeting the specified tolerance by vibration alone ; the best that could be obtained was $\frac{1}{4}$ inch in 10 feet. It was evident that hand finishing would have to be allowed, and this was therefore adopted, although efforts to finish by vibration alone were continued on the outer lane of the first transition curve before being abandoned.

The procedure finally adopted was to produce as good a surface as possible with vibrating screeds and then to go over it with a hand screed to remove the ridges left by the vibrators. Pneumatic compactors were used for the bottom layer. To avoid disturbance of the formwork, rotary electric vibrators were used for the top layer. Test cores showed that a satisfactory bond was attained.

On the steep bays, the finishing process sometimes took as much as 2 hours, four successive passes having to be made with the hand screed. In fact, the speed at which bays could be finished governed the rate of placing. This process required perseverance and much practice ; and the quality of the finished work is largely attributable to the fact that the trained gangs, once formed, were kept together. Particular attention was given to the edges, where poker vibrators were used to ensure compaction and specially shaped arrissing tools were employed. The behaviour of the concrete was extremely sensitive to weather.

The specified tolerance was fully achieved, the actual divergence in most bays not exceeding $\frac{1}{16}$ inch. No artificial making-up, or patch treatment, was required either before or after the concrete had set.

Fig. 11 shows vibration of the top layer of one of the steeply sloping trial bays, and *Fig. 12* shows bottom-layer compaction on the main works with hand finishing of the top layer in the background.

Adoption of hand finishing led to slight loss of surface density and to higher production-labour cost. Vibratory finishing to fine tolerance at high angles might be practicable, given development of special vibrating screeds. The fundamental reason why the concrete tended to slump appeared to be the concentration of energy into the relatively small (3–4-inch) width of the vibrating screed. A wide, heavy screed designed

to vibrate strongly at the leading edge and hardly at all at the trailing edge might prove to be a solution. It is suggested that a square leading edge would reduce the tendency for the surcharge to be drawn under the vibrator more at the centre than at the ends and that a chamfered trailing edge would prevent tearing of the surface. The Authors are, however, confident that the method adopted was the best compromise in the circumstances.

Outputs

Concreting lasted from the beginning of September 1952 to the middle of February 1953. Thus a great deal of the work was carried out during difficult weather conditions and short days. The greatest number of transverse bays concreted in one day was ten (about 120 cubic yards), the average fine-day production being six to eight bays.

Frost

No heating of concrete materials was done and concreting was allowed down to 34° F. on a rising thermometer and 36° F. on a falling. The bays were protected with tarpaulins and straw on frames immediately they were finished and no frost troubles were experienced.

Junction Sections

The new transition surfaces had to be joined smoothly to the existing track. Each of the four junction sections required individual treatment, owing to the differing grades and levels of the existing surfaces. They are all 100 feet long and designed to give smooth change of longitudinal grade. The surface is formed in 1½-inch-thick rolled asphalt on a concrete base grafted into the existing pavement.

North-west Straight

Alternative prices were quoted in the tenders for construction of the North-west Straight in 9-inch concrete or in asphaltic pavement with flexible base. In the accepted tender the former was cheaper, and would have provided useful training for the rest of the work. Unfortunately, steel was rationed at the time, and the saving of 90 tons that could be effected by adopting flexible construction was of great assistance in obtaining the necessary licence. (As it was, 3 months' delay occurred on this account.)

The total construction thickness of 15 inches is made up of :—

3 inches of ashes.
8-inch base course of 1½-inch single-size limestone.
2½ inches of 1½-inch tarmac.
1½ inches of hot-process rolled asphalt.

The base course was originally specified to be of tar-coated stone but, after tenders had been received, the use of a single-size limestone blinded with dust was investigated. This process had been used with success by the Road Research Laboratory in strictly comparative experimental stretches of a main road. Records have shown that this form of base is virtually as stable as tar-coated stone. The price quoted showed a 7 per cent saving on the tar-coated alternative, even though a suitable limestone was only obtainable from about 40 miles away.

Fig. 9 (facing p. 196) shows the $1\frac{1}{2}$-inch single-size limestone being laid. A spreader box drawn by bulldozer was used to spread each of two layers to about 5 inches thickness. Each layer was lightly rolled and then $\frac{1}{8}$-inch-down limestone dust was hand spread on the surface. Further rolling and dusting was done until all the voids were filled and a thoroughly dense base course produced. The resultant base is highly stable owing to the natural binding action of the limestone.

The importance of laying this type of base on a carefully levelled sub-base must be emphasized. Unlike a tar-coated stone base, which can be laid in a partially compacted condition by mechanical spreader, this stone is laid loosely and any irregularities in the sub-base will show as soon as rolling begins. Trouble was experienced because of this and the covering $2\frac{1}{2}$-inch layer of tarmac was eventually laid in two layers $1\frac{1}{4}$ inch thick ($\frac{3}{4}$-inch stone) to help smooth out irregularities.

Drainage

The drainage system was designed on a basis of 1 inch per hour, details generally following those already existing on the airfield. Throughout the inner (lower) edge of the new lengths of track a French drain was provided, using porous concrete pipe and topped with open-texture tarmac flush with the track surface. These drains were connected to the existing system, the latter being improved and extended as necessary.

Most of the drainage was laid prior to construction of the embankments and this is now felt to have been a mistake. A large amount of debris found its way into the French-drain filling, much of which had subsequently to be cleaned out and replaced.

Quantities and Costs

The following is a summary of the final total cost, with some of the major quantities involved :

Clear site, strip topsoil, break out existing concrete, etc.	£15,000
Drainage	£16,000
Earthworks: including 60,000 cubic yards of material in embankments. 35,000 square yards of formation face trimming and sealing. 10,000 cubic yards of unsuitable material excavated, dumped, and returned to borrow-pits.	£41,000
Concrete paving: including 10,000 square yards of 9-inch concrete paving on slopes steeper than 1 in 4½. 20,000 square yards of paving on slopes shallower than 1 in 4½. 3,500 linear feet of expansion joint. 1,700 linear feet of contraction joint.	£59,000
Subway: including access roads	£8,000
North-west Straight, 15,000 square yards, generally level with existing ground	£30,000
Miscellaneous (top seal to bank, guard rail, etc.)	£17,000
Total	£186,000

Conclusions

The basis of design of the surface shape has proved quite satisfactory. In particular, the transition lengths are adequate and the vertical curves scarcely noticeable. However, high-speed driving round these curves needs experience and is not attempted without practice at lower speeds.

Soil tests provided scientific, yet practical, means of controlling the compaction of clay fill so that the embankments can be relied on to support the rigid pavement without significant settlement.

The tolerances set have generally been met. This is particularly true of the ⅛-inch-in-10-feet straight-edge tolerance; in many cases a divergence of not more than $\frac{1}{16}$ inch has been achieved on the transverse bays, and this is greatly to the credit of the contractors, and all their personnel, who are to be congratulated upon the success of their efforts to meet an extremely exacting specification for work which, with the exception of the old Brooklands race track, is believed to have no precedent in the United Kingdom.

The resultant riding quality of the surface is excellent. Tests made with the Road Research Laboratory's "Roughometer" show that the average "inches per mile" on the transverse bays is 95 and on the longitudinal bays 135. These correspond roughly to Profilometer readings of 35 and 64 respectively, and compare favourably with those given by the best concrete road surfaces.

1 in 1½ appears to be the greatest slope at which a reliable concrete pavement can be laid in one operation without resort to top shuttering or subsequent surface dressing.

The Authors are convinced that construction in transverse bays, coupled with high-order accuracy of setting-out, form-shaping, and levelling, is the best method of ensuring a perfect riding surface on super-elevated curves. If they were designing another similar track they would specify construction of the whole track-width in transverse bays. They would space expansion joints at about 50-foot centres for both winter and summer construction. Given freedom of choice, they would adopt a rigid concrete pavement for all parts of the track, flat sections included, in preference to flexible construction. Experience at Lindley indicates that finer limits of surface shape can be achieved with the former, and at lower cost. Maintenance costs should also be lower.

Permanent bench marks have been installed at intervals near the inside edge of the track. Sets of levels have already been taken on the top of the concrete paving and also on the inner 10-foot lane. The predicted consolidation of the embankment itself is $\frac{1}{2}$ inch and of the underlying ground, $\frac{1}{4}$ inch. It is hoped that by the time this Paper is discussed a further set of levels will be available.

Acknowledgements

The works described were executed in accordance with general requirements laid down by a panel of members of the Council of the Motor Industry Research Association and its Director, Mr A. Fogg, M.Sc., M.I.Mech.E. The Authors are indebted to the Association for permission to present the information contained in the Paper, and to Mr Fogg and his staff for their close and understanding co-operation at all times.

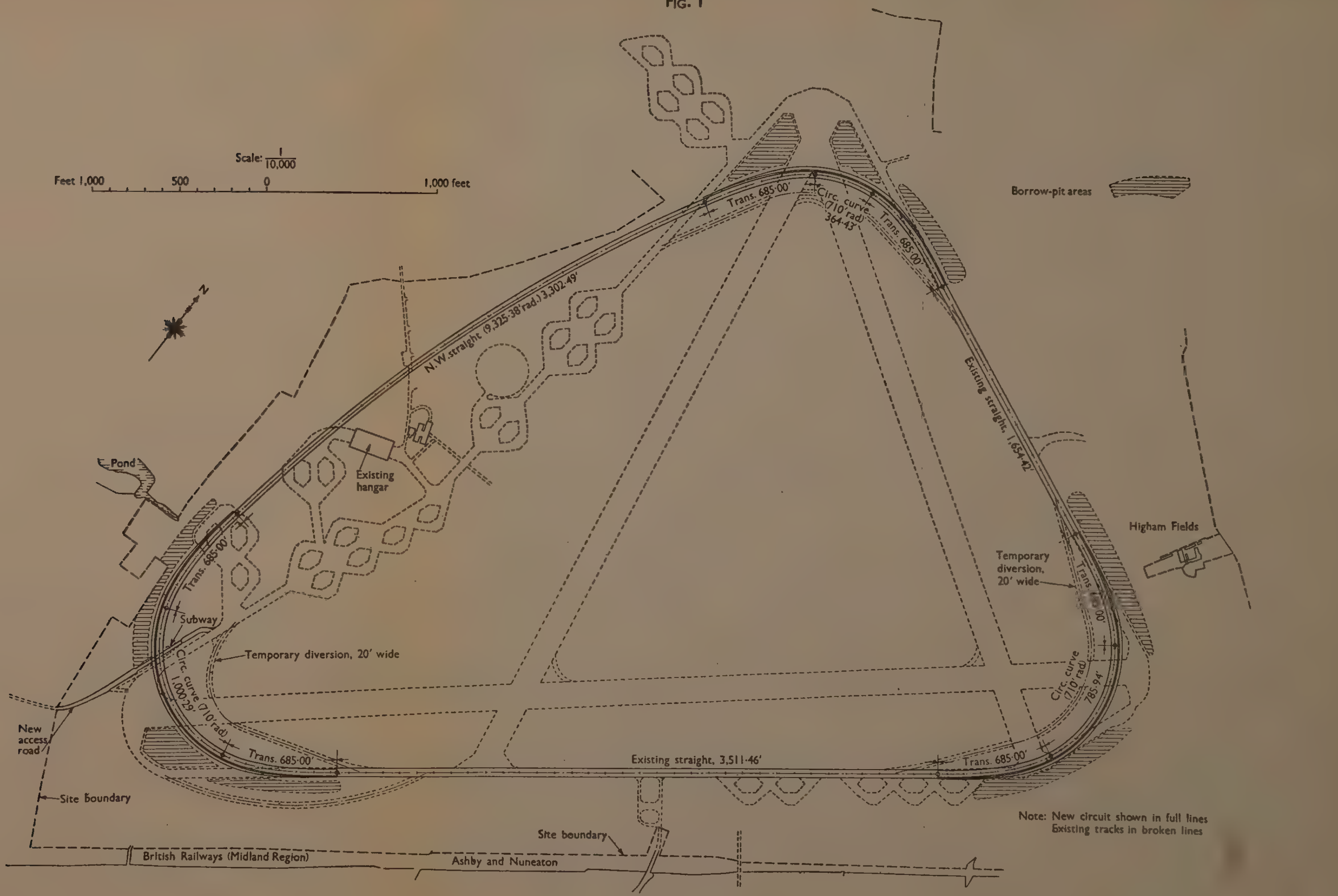
Valuable advice and assistance were given during design and construction by the Road Research Laboratory, for which the Authors tender their best thanks to the Director, Dr W. H. Glanville, C.B., C.B.E., D.Sc.(Eng.), Ph.D., Past-President I.C.E., and members of his staff.

Thanks are also due to Messrs R. A. Stougaard, of Packard Motor Co., and H. H. Barnes, of General Motors, for furnishing particulars of their Companies' tracks in America and for demonstrating those tracks to a member of the Engineers' staff; and to the Director-General, Union Technique de l'Automobile du Motor Cycle et du Cycle, Paris, for similar facilities in connexion with the Montlhéry Track.

The Authors also record their indebtedness to numerous other individuals and firms whose assistance and advice were much appreciated; in particular they would mention Mr W. T. F. Austin, B.Sc., A.M.I.C.E., and Mr B. F. J. Bradbeer, M.C., A.M.I.C.E.

The contractors were Harbour and General Works Ltd, the contract being executed under the responsible direction of the late Mr K. J. H. Evans, B.Sc.(Eng.), M.I.C.E., and Mr W. R. Grigor Taylor, M.I.C.E., assisted by Mr W. L. Mathew, B.Sc.(Eng.), A.M.I.C.E. The Contractor's Agent was Mr J. Oliver and their chief site engineer Mr J. Howard. The Limmer

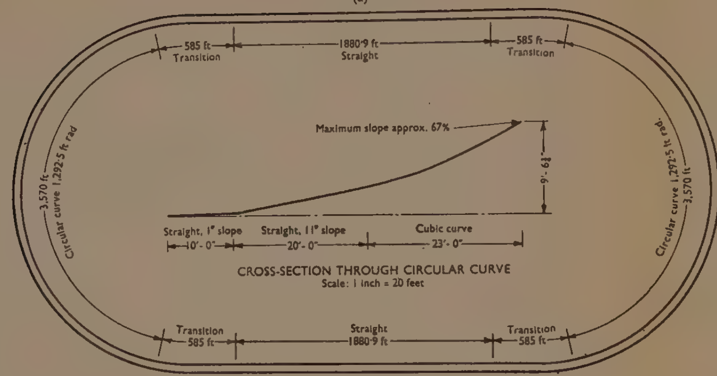
Fig. 1


Scale: 1/10,000
Feet 1,000 500 0 1,000 feet
N
Borrow-pit areas
Trans. 685·00'
Circ. curve (710' rad) 364·43'
Trans. 685·00'
N.W. straight (9,325·38' rad.) 3,302·49'
Existing straight, 1,654·42'
Existing hangar
Pond
Higham Fields
Temporary diversion, 20' wide
Trans. 685·00'
Subway
Temporary diversion, 20' wide
Circ. curve (710' rad) 1,000·29'
Circ. curve (710' rad) 785·94'
New access road
Trans. 685·00'
Existing straight, 3,511·46'
Trans. 685·00'
Site boundary
Site boundary
British Railways (Midland Region)
Ashby and Nuneaton
Note: New circuit shown in full lines
Existing tracks in broken lines

THE DESIGN AND CONSTRUCTION OF A HIGH-SPEED TEST TRACK FOR MOTOR VEHICLES

FIGS 3

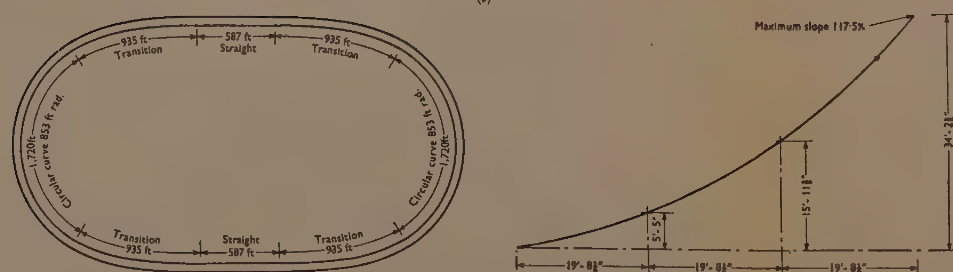
(a)



PLAN
Scale: 1 inch = 1,000 feet

PACKARD TRACK

(b)



PLAN
Scale: 1 inch = 1,000 feet

CROSS-SECTION THROUGH CIRCULAR CURVE
Scale: 1 inch = 20 feet

MONTLHÉRY TRACK

and Trinidad Lake Asphalt Co. were subcontractors for the tarmac and asphalt work. Expansion-joint materials were supplied by Expandite Ltd.

Messrs George Wimpey & Co. furnished cost information which was very helpful in the preparation of preliminary designs and estimates; they also carried out the initial soil survey. The routine soil sampling and testing during construction was done by Soil Mechanics, Ltd.

The Engineers were Messrs Freeman, Fox & Partners, Mr Ralph Freeman being the partner responsible for the work. Mr J. A. Neill, his co-Author and a member of his staff, prepared the detailed designs and was Resident Engineer at the site during construction.

References

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2. E. H. Davis, "Pavement Design for Roads and Airfields." Road Res. Tech. Paper No. 20, H.M.S.O., 1951.
3. W. H. Glanville, A. R. Collins, and D. D. Matthews, "The Grading of Aggregates and Workability of Concrete." Road Res. Tech. Paper No. 5, H.M.S.O., 1947.
4. "Design of Concrete Mixes." Road Note No. 4, H.M.S.O., 1947.

The Paper is accompanied by nine photographs and six sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Discussion

Mr W. R. Black said that he would not attempt to comment on the technical aspects of the high-speed circuit, but, as President of the Society of Motor Manufacturers and Traders, his interest in the installation at Nuneaton—the research station, the proving ground, and the high-speed circuit—was naturally very considerable. The Motor Industry Research Association (known as MIRA) was a fine example of how highly competitive individual companies could co-operate in their joint interests on a national scale.

About 7 years previously the Society of Motor Manufacturers and Traders had assumed responsibility for the Research Association, which had previously been controlled by the Institution of Automobile Engineers. They undertook to use their best endeavours to ensure an income of £40,000 a year for 5 years, which would enable them to qualify for a Government grant. It was assumed that about one-third of the grant-earning income would come from outside sources, such as operating companies and nationalized concerns, and that their own industry would find the remainder. At the same time a new governing council was formed,

two-thirds of its members being nominated by his Society and the remainder representing outside interests. By that means continuity with the old organization had been maintained.

Prior to that date, 7 years ago, some members of his Society had individually contributed to collective research, but as a Society they had taken no part in it, and it had been with some reluctance that they had assumed the responsibility of finding a sum of approximately £27,000 a year. The scheme, however, captured the imagination of their members, and at least double that amount had been forthcoming every year. In addition, two special appeals had been launched, each of which had brought in from their members more than £100,000.

It had soon become evident that the original premises which had been inherited from the original organization were inadequate, and the present site at Nuneaton was selected. There were now modern laboratories, proving grounds, cross-country tracks, water splashes, corrugated and pavé roads, dust tunnels, and last but by no means least, the fine high-speed circuit which was the subject of the Paper. Without it, it would be necessary to test their vehicles on the woefully inadequate roads of Great Britain, to the danger of life and limb not only of their drivers but also of the general public. MIRA, with this high-speed track, was now playing a major part in the struggle to ensure that the motor industry remained Britain's chief exporter; they could now reproduce conditions comparable to those which their products would have to meet throughout the world.

Mr W. T. F. Austin said that at the time that the Lindley track was being designed he had been working in America, and at Mr Freeman's suggestion he had visited Detroit. The General Motors and Packard firms had been very helpful, showing him their tracks, indicating the good points, and, what had been even more helpful, describing the defects; they had criticized the proposals for the MIRA track in a very friendly and helpful manner.

General Motors at that time were operating an old track (the first speed track ever made for testing cars) which had been almost destroyed by tanks during the war; they had decided to rebuild it because it was obsolete. The balanced speed, without tire friction, was only 85 miles per hour, the radius of the curves was too short and the transitions too short, most of those defects being the result of poor lay-out arising from the use of existing roads. Packard, on the other hand, had also had their track destroyed by tanks but had rebuilt it along its former lines, because they had found their track entirely satisfactory. When building their track they had benefited from the early experience of General Motors.

Mr Austin then showed some slides illustrating the reconstruction of the Packard track. A plan of the track showed that it had the ideal shape, giving the maximum radius of curvature consistent with providing some straight. The transitions had been made in virtually the same way

as at Lindley, but, as the Authors had mentioned, the circular curves, which were very much longer, had been finished by means of a machine made specially for the job. It consisted of a beam to which were fastened a number of plungers which rotated backward and forward through 180 degrees and at the same time moved up and down; it seemed to knead the concrete into the correct form. He did not see the need for such a complicated machine, but it had produced a track of the same quality as that at Lindley. (He had travelled round both tracks, and on both the surface of the concrete was excellent.) The finished curves were very similar to those described and illustrated in the Paper.

The General Motors engineer had said that they certainly could give advice on how not to build a speed track, and described also how they proposed to rebuild theirs. On the other hand, Packard said "If you want a good track, build a duplicate of ours; you will never regret it." That would have required a great deal of money, but for MIRA an excellent track had been produced very cheaply.

The Americans would not consider using tire friction. It should be remembered that the climatic conditions at Detroit were worse than those at Lindley. They much preferred to have a balanced speed, and they liked to have a little excess superelevation as a safety margin. On such a cross-section if a car got out of control and tended to go too far up the banking, the excess superelevation threw it back to its speed lane; if it went too low the excess centrifugal force threw it back to its speed lane, whilst even if a car skidded sideways-on it still naturally went to its proper speed lane. On all those tracks it was possible to take the hands off the steering wheel and the car would steer itself round the curves. Even on the wrong speed lane the car would go up the bank and down again and eventually find its proper lane. The Americans were quite sure that if a car were driven round the track in the wrong lane or at excess speed using tire friction, the tires would be worn out in 2 days.

Mr Freeman mentioned 180 miles per hour as the maximum speed at which it should be possible to drive a car round the Lindley track. On the day that the Packard track was opened a racing driver was invited to drive round it. It was a 100-mile-per-hour track, and the driver made sets of three runs (one accelerating, one at speed, and one decelerating). His speed was 149 miles per hour. The tires were changed after every run, but whether that was necessary or not Mr Austin did not know. He thought that the corresponding speed on the MIRA track, with tire friction and superelevation having approximately equal effect, would be about 120 miles per hour; at the time of the test on the Packard track, however, the guard rail had not been installed and the driver had almost gone over the edge, so the performance had not been repeated.

He would like to express his gratitude (and he felt that all concerned would agree), for the helpful manner in which the Americans had provided all the information they could give on the subject of their proving grounds.

He had one question to put to the Authors. The transition curves at Lindley were designed for a rate of gain of centripetal acceleration of 4 feet per second per second per second, but the normal figure on roads was much smaller—about 1·5. He had not noticed any unpleasant sensation in going round those curves, but his sensitivity in that respect had been ruined by riding on London buses, and he would like to know whether others who had ridden on the Lindley track had noticed any discomfort. He would have no hesitation in shortening transition curves if it would suit the road to do so.

Mr W. R. Grigor Taylor referred to the question of choice between smooth rollers and rubber-tired rollers. His own firm, as contractors, had been mainly concerned on the embankments with making sure that they were kept as clean and tidy as possible, because the setting-out had been one of their biggest problems throughout the work. They considered that a smooth-tired roller would give them a cleaner bank than would a rubber-tired roller.

Trimming the embankment entailed taking 18 inches of compacted soil (about 10,000 cubic yards) off the completed bank; that was a major operation and so mechanical plant had been used. The main problem had been that the embankment was curved in three directions and there had been the greatest difficulty in controlling the setting out as the trimming was done. It had been impossible to set the fairly elaborate screeds used for the paving work, because they were 18 inches down from the surface which had to be trimmed. It had therefore been necessary to devise a system of temporary screeds, readily movable, so that the scrapers could operate. Those caused them a great deal of trouble, and much credit was due to the engineers who watched the setting out.

The timber screeds (templets) had also been rather unusual. On the transition sections of the curves it had been necessary to construct about sixty screeds, each one of a different curved section, and to an accuracy of 0·005 foot in order to restrict the surface undulation to specified limits. The floor of a shed had been adopted as a setting-out base, and the screeds were marked out from a superimposed set of piano wires. The fabrication of the screeds did not involve great difficulties, nor did the setting of them, although that had been a laborious job. The changeable weather, however, alternating between hot sun and showers, had caused trouble and the screeds had shown a marked tendency to warp—sometimes to the extent of an inch or more—between the stages of setting up and concreting. It was as well that the warping was noticed at that stage, for it was then possible to take out the warped screeds and reshape them in the setting-out shop before re-setting. The tendency to warp was probably attributable to the fact that the screeds had had to be made from 9-inch-by-3-inch timbers by cutting both with and across the grain, so that the remaining pieces of timber were left in a very unstable state of tension.

The Paper had dealt exhaustively and accurately with all the problems which had been encountered in obtaining the specified finish of the concrete, particularly on the steeply-sloping parts of the track. Mr Grigor Taylor, however, took exception to the use of the word "compromise" on p. 201. Presumably the Authors used that word because on the steeper sections it had been essential to finish off the concrete by hand. The Contractors had devoted a good deal of time to that problem, and had been helped by Mr Neill, but it had to be admitted that they could not get the finish which was wanted with any method of vibration, and had had to resort to hand screeding. Mr Austin had shown a slide of a rather elaborate tamping arrangement used in the making of the Packard track, but in the background of that picture could be seen men screeding off with hand screed boards.

Mr Grigor Taylor believed, therefore, that there had been no question of a "compromise" in the case of the Lindley track. He considered that the word "compromise" was an odious word and was used far too much by everyone. In the present instance there was in his opinion no question of "compromise," and he felt the word should be deleted. The sentence he objected to would then read—"The Authors are, however, confident that the method adopted was the best in the circumstances."

Mr B. F. J. Bradbeer commented upon the open-minded and enlightened attitude of the Authors in gathering information from all possible sources on points of theory and practice before drawing up their specification.

He had been very interested in the methods adopted to ensure that the requirements of the specification were in fact carried out, notably with regard to the earthworks, where in particular the engineers had realized the necessity for full control by sampling, selection, compaction, and temporary protection of the finished earthworks until such time as they could be covered by the load-bearing structure. It appeared that the engineers had been conscious of the necessity to pay the contractor for those various operations in such a way that there was little reason for the contractor to seek to avoid his obligations so far as those details were concerned. Such arrangements seemed to be a great improvement on some former specifications of which many members present would be aware, whereby contractors had often been left to make their own examinations on the site and to carry all the risk. The work described was, Mr Bradbeer suggested, an example of the fundamental importance of the earthwork responsibilities being accepted courageously by the engineer, who should so arrange his specification and bill of quantities that the contractor was clear beforehand what he was required to do and was aware that he would, in fact, be paid to perform those operations in the proper manner. For example, if soil was to be selected and transported in various directions to make the best use of it, or if some of it was to be discarded as useless, the bill of quantities should be so drawn up, as

the Authors appeared to have done at Lindley, with the different operations separately set out, so that they could be intelligently priced and measured. Further, Mr Bradbeer would make the suggestion, from personal experience, that there was much merit in the billing of the various kinds of plant required for the compaction of earthworks at day-work rates. In that way the engineer could have complete control of that most important feature and could apply the various machines as might be necessary, using them when and where necessary to obtain the optimum compaction.

He had also been very impressed by the provision made in the contract for trials of the various methods, such as trimming the formation, design of concrete mixes, and placing and finishing of concrete to fine limits—all done on a site separate from the finished work. In that way, engineers, contractors, and workmen seemed obviously to have gained confidence and skill in the methods to be adopted, and no part of the finished structure suffered from initial mistakes, "trial and error," or lack of experience. One could sense that out of such arrangements, and out of the perseverance and practice required during the trials, there had developed a valuable team spirit, and that the high quality of the finished work was, in great measure, attributable to keeping together the teams of engineers and workmen who had been trained in that way.

Mr R. R. W. Grigson said that it had needed considerable courage on the part of the Authors to attempt to lay concrete at such an angle as that shown in *Fig. 12* (between pp. 196 and 197). There was a reference in the Paper to the old Brooklands track. He believed that Brooklands had been laid about the year 1912, and he wondered whether the Authors had been fortified in their resolution by reading some Paper on the Brooklands track. Very much less could have been known about concrete in 1912 than was known to-day, but the engineers responsible for Brooklands had also not been lacking in courage, because probably the slopes there were comparable with the slopes at Lindley.

Both the Authors and the Contractors were to be congratulated on producing an excellent job. The riding qualities in particular struck him as being exceedingly good. Unless Mr Neill, who had been his escort, had picked a particular track to take him over, he would say that the riding qualities compared favourably with those obtained on first-class work laid on the flat, yet the Lindley track had been laid at the extraordinary angles shown in the Paper.

One of his first impressions had been that the transition curves seemed to be extraordinarily short for the degree of superelevation imposed, but he agreed with a previous speaker that they created no discernible discomfort. He thought that road engineers should re-orient their views on the question of the length of transition curves. They had in the past been aiming at curves which were far too long, and long curves cost money. When the Ministry of Transport had in mind the revision of their circulars

relating to lay-out they should look at the Lindley track from that point of view.

It was made clear in the Paper that the Authors had treated clay as a structural material. They had treated it with respect, and not as so much muck, which was very often the attitude on public works. They had mentioned the rejection of some material because it had been too dry. It was refreshing to know that in Britain clay could sometimes be too dry for such work, but it had occurred to Mr Grigson that, instead of rejecting it, it might have been cheaper to put the 13 per cent of moisture back by spraying it. Had that been considered ?

The design wheel loads at the inner edge were 8,500 lb., falling to 2,000 lb. at the outer edge, yet the same thickness of concrete had been adopted throughout the slab. He would have thought that such a sharp reduction in design load might be accompanied by a comparable economy in slab thickness.

Some of the work done by the Road Research Laboratory on the macadam base had been done in Mr Grigson's county. The fines had been vibrated into the stone by means of a 1½-ton German "Vibromax" compactor. That seemed to be an excellent way of constructing such bases. Had the Authors considered using it and, if so, would it have affected the thickness or the design in any way, compared with the method which they did use, which he understood was sweeping-in the fines ?

The top course of the concrete had been laid after the bottom course, and yet the Authors had stated that excellent bonding resulted. What had been the maximum interval of time between the placing of the bottom course and the placing of the top, to ensure such good bonding ?

Finally, Mr Austin had shown that the Americans, in spite of the fact that they always referred to "pouring" concrete, could produce as dry a concrete as was produced in Great Britain when it became necessary to do so.

Mr Albert Fogg observed that he had been driven round, or had driven himself round, the General Motors track, the Ford track at Dearborn, and the track at Montlhéry, and could say without hesitation that the track produced at Lindley was at least as good as, if not better than, any of those.

He referred to the possibility of trouble arising from the equal spacing of joints in the concrete. There would inevitably be some small periodic impulse applied to the suspension of the motor vehicle, and at some speed those impulses might correspond with the resonant frequency of the sprung mass. The point had been considered, but equal spacing had been favoured and, perhaps because of the very great emphasis placed on the finish, the track had such minor discontinuities in it that no such periodic impulses could be discerned when driving round the track. On the other hand, he wondered whether perhaps it might not have been rather easier to produce a less accurate finish, but with unequal spacing of the joints in

the concrete. It was a somewhat academic point, because the track was perfectly satisfactory.

He went on to discuss the need for a test track such as that at Lindley. At the present time there was perhaps a tendency to assume that everything could be done in the laboratory. A good deal, of course, could be done there, but since the war the British motor industry had been exporting so many vehicles that design was determined almost entirely by those considerations. In countries such as the United States and Canada, quite small cars were driven at maximum engine output for hour after hour, and there was not enough experience available in British design offices as to how such factors as heat transfer from engine to coolant and coolant to air were affected by such conditions, or of the range of temperature at which the transmission operated. Such things were extremely difficult to determine in the laboratory, and it must therefore be assumed that a test track would be valuable, not simply for the high-speed cars, but for production cars whose top speed was only 70 to 80 miles per hour. It would make them much more reliable, and the background of the designer's experience would be improved by use of such a track.

Mr Fogg confirmed what other speakers had said about the quality of the surface of the Lindley track and it was his experience that that applied even at the top of the 34-degree banking. The use of the track was still largely experimental but useful experience had been gained. He had himself driven on the track a good deal and felt quite confident traversing a complete curve, including the transitions, at speeds up to 90 miles per hour without touching the steering wheel. Although Mr Freeman's estimate of 180 miles per hour was, he appreciated, a purely ideal consideration and something not to be achieved in practice, the circuit had been lapped at more than 130 miles per hour; the driver who did that said that he found it quite a simple experience compared with driving on some of the racing circuits of Europe.

The question of traffic on the circuit was one of the main problems. The Association had a very large number of members and it was quite common for twenty to thirty different firms to use the proving ground at the same time. They had therefore to exercise some very serious control over the traffic, because they could not afford to have accidents. At the present time the track was divided into two lanes, an inner lane with a maximum speed limit of 75 miles per hour and an outer lane with a minimum speed limit of 75 miles per hour. The outer lane was restricted in numbers much more severely than the inner, and if any car exceeded 100 miles per hour the number of vehicles on that track was limited to three and overtaking in the outer lane at the bends was forbidden. So far they had operated very successfully.

Referring to the guard-rail, Mr Fogg said that so many opinions had been expressed that there had been some difficulty in deciding finally what to do, but he thought that what had been done was an excellent

compromise. There was still the odd driver, however, who thought it a menace, and any alternative suggestions for some psychological or physical barrier would be welcomed.

Mr D. H. Little said that what appealed to him most about the Paper was that it was so eminently sensible. The engineers had been faced with an unknown problem of placing concrete on steep slopes so they at once arranged to have experimental work carried out and, after those experiments were finished and an agreed procedure had been arrived at, they did not hesitate to make further adjustments as even more experience was gained ; when the specification could not be carried out they admitted it and made arrangements accordingly. It seemed obvious from the Paper that that sensible attitude of give and take applied throughout the whole job, and he felt that that must have been so to permit the work to be done in little more than a year, which was a remarkably short time.

Another significant feature of the Paper was the generous acknowledgements made to those who had assisted with various specialized items of the work. A builder friend of his was developing a housing estate near him, and most of the work—brick-laying, plastering, roofing, painting, and so on—seemed to be done on sub-contract, so that he wondered what the builder did himself. Those engineers who worked with architects probably felt the same about them at times. The fact was, however, that every phase of life was becoming so complicated that everyone had to rely more and more on specialists. Clients were themselves often technical or had strong technical departments, and that could mean that the preliminary study stages could be long and tedious.

The Authors said that tenders were invited in November 1951, and Mr Freeman had said that his firm was appointed in May 1949, so that it took more than 2 years to get the job designed and considerably less than 2 years to build it. That was in accord with his own experience, that nowadays such works, especially when scientific study was involved, did take a very long time to prepare.

Referring to drainage, Mr Little said that at Naval air stations, unless exceptional conditions obtained, the drainage was based on 1 inch of rainfall per hour. French drains with open-texture tarmac had been used during the war, and, as at Lindley, soon clogged up ; maintenance engineers inserted gullies at the low spots, which showed themselves by ponding. In all post-war work shallow concrete channels 5 feet wide and about 2 inches deep had been provided with gullies at 50 to 150 foot intervals.

The building of the 8-inch flexible road foundation with numerous layers of 1½-inch stone was quite new to him. Did it give as good a result as 6-inch hardcore or hand pitching ? He would have thought that in the rolling of the small stones one would be rolled on top of the other, and he would like to know more about that. The design of paving slabs was a fascinating subject. Could the Authors say how to judge the thickness of a piece of concrete capable of carrying a wheel load of 8,500 lb.? From

the description of the concrete he thought that a minimum cube strength of 4,000 lb. per square inch would be obtained with a failing tensile stress of 600 lb. per square inch. If twin slabs could have been used to eliminate corner effects, theoretically two thicknesses, each of 3 inches, would have been enough. He did not suggest that anyone would build roads only 3 inches thick, but he had built them 6 inches thick with two layers, taking loads of 18,000 to 20,000 lb. That question of road design, when on twin slab work, resolved itself into a question of just how thin one dared to make the individual slabs.

Would the Authors give details of the dowel bars at expansion joints and of the method of making the other joints "fully tied?" He did not quite appreciate what a "fully tied joint" meant. Would the Authors also enlarge a little on the need for the mesh reinforcement. With a failing stress of 600 lb. per square inch a plain concrete section 9 inches thick had a resistance moment of 8,000 lb.-feet per linear foot. Even at a steel stress of 60,000 lb. per square inch a reinforced slab with about 0·1 per cent reinforcement had only half that strength; so, as with all roads, reinforcement at only 8 lb. per yard seemed to be useless from the structural point of view. If it was placed 3 inches below the surface, would it really help against surface cracking? Experience with mass concrete on runway construction was that high-grade concrete would not crack if it could slide freely in the initial stages of drying out and if it could continue to do so under the effects of subsequent temperature changes.

Mr Little had also been interested in the implication that a better running surface could be obtained with concrete than with tarmac. One speaker had said that the surface at Lindley was better than that of the average concrete road. The average concrete road which Mr Little had been over in a car gave him an up-and-down sensation as though he were on the sea, and he had always been glad to come to a stretch of tarmac. He had used tarmac on runway construction, and had had to make up fairly deep depressions, so that in some places the tarmac was 8 inches thick. All of it had been laid with a Barber-Greene machine and finished to a tolerance of $\frac{1}{8}$ inch in 10 feet. He felt certain that a surface of that nature would be a better running surface than anything which could be obtained with concrete.

Mr A. A. Osborne, who said that his interest in the Paper lay chiefly in the practical application of soil mechanics, recalled that there had been a time when the use of clay in embankments had always been considered with reluctance. The Authors had tackled the job in a manner compatible with the principles of soil mechanics but might have given more soil data in the Paper. Could some comparative dry densities be given—dry density in situ for the clay in borrow-pit, and after compaction, and for the Proctor test of the reconstituted clay? Could the Authors give the moisture content of the 10,000 cubic yards of clay which had been found to be

unsuitable because it was too dry? it would also be interesting to have a mechanical analysis of the clay.

The whole question of soil mechanics in its practical application boiled down to costs in the field, and from Mr Osborne's experience, mainly in road construction, many contractors had not had experience in constructing earthworks to modern specifications. A contractor who had done an excellent job for Mr Osborne 2 or 3 years previously, claimed that he had lost £5,000 on the embankment. Mr Osborne had suggested to him that he had not sufficiently realized what it cost to compact earthworks with modern equipment to a maximum density and what were the potentialities of a modern earthworks specification.

On p. 203 a total sum of £41,000 was given as the cost of earthworks, and it would be useful to have that broken up. If the 60,000 cubic yards of material in embankments were taken at 10*s*., the 35,000 square yards of formation face trimming and sealing at 5*s*., and the 10,000 cubic yards of unsuitable material at 4*s*., that would come to roughly £41,000, but Mr Osborne realized that his calculation might be unreliable because there were so many factors of which he knew nothing. If the Authors could, without prejudice, give figures of the cost of compacting and dealing with earthworks to modern specifications it would be a considerable help not only to those responsible for design but to contractors, many of whom went into such work without enough information.

He agreed with Mr Bradbeer that specifications and bills of quantities should include more detailed information on the use of various plant; it would help the people responsible for supervision and it would also have the advantage of giving a much keener price. A contractor might either not go into the job sufficiently and lose through ignorance, or play safe; in the latter case the job would not be so economical as it might have been.

Mr Neill had invited comments on dry density comparisons. The Ministry of Transport specification laid down 95 per cent of the maximum dry density which could be obtained with the most effective plant at the specified moisture content. Mr Osborne did not think that that was fair and thought that the specification should bear some relation to a laboratory test as a guide.

It had been stated that there were three soil mechanics engineers on the Lindley job, with three assistants; that was a considerable staff and, he thought, justified the inclusion of more soil information in the Paper.

The work had been done between July and September, an ideal time of the year, but to the best of his recollection it had been rather wet. Had the contract been timed and designed so that those earthworks had to be done between July and September, or had the Authors simply been lucky in that respect? The cost of doing the same work in the middle of the winter would have been much greater. Such contracts should be designed to start and finish at the right time of the year.

Dr A. R. Lee asked the Authors to say a little more about the

finish which they obtained on the straights with a flexible construction. They gave no information in the Paper about the regularity of finish, and he wondered whether they had that information. There was no doubt that for the curved ends the form of construction used was the best, and they had achieved a remarkable degree of regularity. In some cases they were within $\frac{1}{16}$ inch over a 10-foot straight-edge. That had been done by hand-laying the concrete. On a straight, over lengths of 3,000 feet, presumably they would lay their concrete by machine. The laying of concrete by machine required a different technique and different degrees of precaution had to be taken to get anything like the degree of finish which they had achieved on the ends. In laying the flexible construction it was a matter of common experience with modern machinery to get very high degrees of finish.

In answer to the question asked by Mr Neill, when introducing the Paper, as to how the density of the soil should be expressed, Dr Lee said that the aim in soil compaction was to eliminate the air from the soil at the particular moisture content at which the compaction work was carried out.

Mr Neill had also asked a question about rollers, and had shown that good results had been obtained by using the smooth-wheeled roller. On the other hand, Dr Lee said, when compacting a soil having a lower degree of stability there was an advantage in using pneumatic-tired rollers.

Turning to the question of broken stone *versus* hand pitching, recent experiments had shown that a base of more uniform strength could be produced with broken stone than with pitching, and that broken stone had the advantage that it could be compacted to a greater degree initially, thereby resulting in less settlement and less deformation under traffic.

Colonel Sydney Green called attention to the statement at the end of the Paper in which the Authors expressed the hope that by the time the Paper was discussed a further set of levels would be available, and said he would be interested to know whether there had been any movement in the structure.

Mr W. A. Lewis said that Mr Neill, in presenting the Paper, had raised one or two points on compaction and asked what was the value of specifying the state of compaction to be attained in terms of a relative compaction, that was, in terms of the maximum dry density achieved in the laboratory compaction test? Mr Lewis said that the Road Research Laboratory considered that compaction should not be specified in that way, because in the laboratory compaction test the energy input did not necessarily bear any relationship to the energy provided by the compaction plant in the field. To specify the compaction to be obtained in the field in terms of an arbitrary amount of compaction in the laboratory was, they felt, illogical. They thought that the better plan was that which the Laboratory had recommended to the Authors and which had been adopted, namely to specify a limiting permissible air content in the soil after com-

paction. If the soil type or moisture content varied on the site, difficulties occurred when specifying a density or relative compaction which were avoided if the state of compaction were specified in terms of a limiting air content.

The Authors, in reply, thanked Mr Black for giving a brief but comprehensive outline of the history of the Motor Industry Research Association which had helped to put the story told in the Paper into proper perspective.

Mr Grigor Taylor seemed to have regarded the use of the word "compromise," at the end of the section of the Paper dealing with concreting, as in some way a reflection on the operations of the contractor. That had certainly not been the intention of the Authors. It was a fact that the Engineers had originally specified concrete finished entirely by vibration, but they had satisfied themselves on the job that it was not possible to do that and at the same time produce the accurate surface required. By agreement with the contractor, they did therefore adopt a slight change in the specification for that work. The Authors considered that that change amounted to a compromise, and they did not propose to alter that word, but they thought it had been a successful compromise.

The Authors had tried to make clear in the specification what they wanted the contractor to do, because they realized that the job was not an easy or straightforward one, and Mr Freeman thought that they had succeeded in some measure in doing that. At any rate the contractors had begun the job with a clear idea of what they had to do, and they did it extremely well; they deserved the greatest credit for all that they did, and not least for the way in which they managed their labour and developed and maintained the team spirit amongst their workmen. Without that, the job would not have been so good as it was, particularly bearing in mind that so much of it had been hand work.

Mr Freeman did not quite like Mr Bradbeer's suggestion of having the operations of different types of plant paid for at day-work rates. Perhaps one could and should have rates quoted for work done by different sorts of plant; for instance, if there was a rate for compaction of soil, or for placing soil in position and compacting it, the tenderer might be invited to quote alternative rates for the use of different types of plant.

Mr Fogg had raised the question of the spacing of the joints and resonance. Mr Freeman wished to make it clear first of all, in case there should be any misunderstanding on the point, that the only joints on the curves which could develop resonance were the expansion and contraction joints which occurred at 50-foot intervals. There was no appreciable joint in the surface between the separate slabs of which the pavement had been constructed. At 100 miles per hour a 50-foot spacing corresponded to a frequency of $\frac{1}{3}$ second. He believed that the frequency of the unsprung mass of many cars was of the order of $\frac{1}{12}$ second. That would be about the frequency of crossing the construction joints between the individual slabs, but he did not think that those were sufficiently noticeable or caused

sufficient change in the surface to develop vibration. It would have complicated matters in many ways if it had been necessary to space those joints irregularly, and it would have complicated matters a good deal if they had decided to construct a rougher surface. It was one thing to specify a smooth surface and get it, but quite another to specify a surface with a certain degree of roughness and make sure that one got just that degree, no more and no less. He also thought that one wanted a smooth uniform surface on a concrete road. Irregular roughness might cause hammering, which would in due course tend towards the breaking up of the edges of the slabs at expansion joints, and other troubles of that kind.

Mr Fogg had mentioned that he had driven round the Lindley track at 90 miles per hour with his hands off the wheel. According to the Authors' mathematics that was theoretically impossible, and Mr Freeman could only assume that Mr Fogg had really been doing 84 miles per hour and that his speedometer had been of the order of 7 per cent fast.

Mr Fogg, intervening, said he must correct that. He had used a calibrated speedometer. Perhaps the wind had been in his favour.

The Authors, commenting on Mr Little's remarks, said that they did not want to give the impression that the whole of the job had been a question of give and take, because that had not been so. The Authors had reckoned that, with the exception which they had mentioned, they had had a clear and complete specification before they started, and both they and the contractor had adhered closely to it.

With regard to Mr Little's remark on concrete roads in general, the Authors thought that that was an unfortunate generalization to make. There were plenty of bad concrete road surfaces, but there were some good ones as well, and he recommended Mr Little to try a section of the A22 road (London to Eastbourne), near a place called Lower Dicker, where there was an excellent surface for quite a long stretch. The surface at Lindley was about equal to that; it might be a little better, but then it was much newer. Better results were undoubtedly obtained on the 10-foot straight-edge on the concrete pavement than had been obtained with the Barber-Greene machine on the flexible pavement. Whether that was because of any irregularity in the base under the flexible pavement the Authors did not know. They believed that they got the best that the Barber-Greene machine could give, but it was not so good as hand work on the concrete.

In reply to Colonel Green's request for settlement data, the Authors said that they had taken some levels recently, after an interval of 6 months, bearing in mind that most of the embankments had been laid about a year ago; in other words, they had been dealing with the second half of the period over which most of the settlement was going to take place. Those levels were, in their view, quite satisfactory, because they showed in general a maximum drop of about $\frac{3}{8}$ inch at the top of the track, accompanied in some cases by a rise of up to $\frac{1}{8}$ inch at the bottom, which might have been

caused by swelling of the clay or by the slight settlement at the top of the embankment.

Mr Austin had mentioned that the Packard track had a design speed of 100 miles per hour balanced, which was very nice indeed. Unfortunately the site at Lindley was limited and the track had to be fitted into its confines, and, though the balanced speed was only 84 miles per hour, it seemed quite satisfactory in service. Mr Fogg had mentioned that the maximum speed to date had been more than 130 miles per hour.

With regard to the lengths of transition curves, the rate of change of centripetal acceleration was 6·5 feet per second per second per second at 100 miles per hour. At 90 miles per hour, although there was no discomfort, one knew that something was happening. A figure of 6·5 feet per second per second per second would not be satisfactory for normal roads, but the figure of 1·5 which was very often adopted was unduly conservative.

The Authors had not said very much about the setting out of the screeds and fabrication, and were very glad that Mr Grigor Taylor had dealt with that important point. Warping had occurred, and the fifty-one screeds on the transitions had suffered by the end of the job; some planing off or building up had had to be done after the screeds had been placed in position.

Mr Grigson had mentioned a Paper on Brooklands. The Authors did not know whether there was such a Paper and certainly had not referred to one, but the slopes were of about the same order. Mr Neill had not chosen the best piece of track to take Mr Grigson on; there was nothing to choose between the three lengths of concrete pavement. With regard to dryness of clay and spraying, the Authors had considered the possibility of spraying at one stage, but had rejected it. The difficulty was that, whether dumped and rejected or dumped, sprayed, and placed in embankments, double handling would have been necessary. The fact that the operation of spraying would need careful control would have made it a more expensive proposition than the complete rejection specified.

On the question of the thickness of concrete, the slab could have been thinner at the top, but there were certain objections. The formation and transverse templet forms would have had to be formed to a stepped cross-section. The top of the bank might settle, and it was advisable not to reduce the strength of construction towards the higher parts. No speaker had mentioned the absence of a base course. It had been a major decision to have no base course under the lower part of the track. That was believed to be in line with modern thought which regarded the omission of a granular base course on clay sub-grades as satisfactory provided that certain precautions were taken. At Lindley the omission of the base course had been compensated by the application of the bitumen emulsion to the formation and also the provision of strips of damp-proof course below the joints, in addition to normal sealing of expansion and contraction

joints. However, the fact that no base course was provided had led to a certain conservatism in slab design.

The point about the German compactor was an interesting one. It was probably an excellent machine, but in fact the main difficulty on the North-west Straight had been spreading the fine limestone, which had been a slow and laborious business. A smooth-wheeled roller was undoubtedly effective, if slow, in forcing the fine material into the voids. The rate of rolling was perfectly consistent with the rate of spreading, and it was doubtful whether the compactor mentioned would have contributed appreciably to the speed of that part of the work.

The question of bond between the two layers of concrete was important. Test cores had been taken from the concrete laid in the trial length. That particular slab had been laid under somewhat unfavourable conditions. It had been a very hot dry day and an interval of $2\frac{3}{4}$ hours had elapsed between finishing compacting the first layer and the start of laying the second. Four cores had been taken and there had been no sign of parting at the reinforcement. On the job the second layer had usually followed within an hour.

Mr Little had asked a number of questions. The Authors thought that a porous concrete channel might have formed a better finish for the French drains than coarse tarmac.

It had been difficult to judge the thickness of concrete slab necessary. Something much thinner than 9 inches would, in all probability, have proved satisfactory, and they understood that slabs down to 4 inches thick had recently been laid and had lasted already for some years. No one could say how long they would last, but they had already lasted 3 or 4 years with no trouble at all, and for heavier loads than on the Lindley track. At Lindley, however, the thickness chosen was undoubtedly safe, and that was the important point, bearing in mind that it was on a clay embankment. The reinforcement was not of value from the structural point of view, and the main object had been to prevent cracking. The use of 8 lb. per square yard reduced cracking to very minor proportions. Dowel-bars at expansion and contraction joints were 1 inch in diameter and 2 feet 6 inches long at 1-foot centres. Tie-bars at fully tied contraction joints provided 2 square inches of steel for every 10 feet run of joint; that was to say, they were $\frac{1}{2}$ inch diameter, 3 feet 6 inches long and spaced at 1-foot centres. Dowel- and tie-bars were at the mid-height of the slab.

With regard to the finish on the tarmac on the North-west Straight, the main factor which made the North-west Straight compare unfavourably with the concrete was the long wave effect. The local surface was very good, but the long undulations affected cars travelling at the higher speeds.

Mr Osborne had asked for figures of comparative dry densities. The Authors could not give absolute figures, but a 95-per-cent degree of compaction had been more than attained in about 90 per cent of cases.

In dealing with plant, the Authors had mentioned the fact that for each scraper one roller was required. Contractors probably knew how much material a scraper could put down, and if adding the cost of a roller to that would give them something to go on. The contract had been planned originally so that the earthworks could be done between April and June 1952, but the three months' delay in the issue of a steel licence made it later. As it happened that did not matter, because the weather had not been too bad.

Dr Lee had answered very satisfactorily Mr Neill's question about relative compaction, and had made some references to machine-laid concrete with which the Authors agreed.

Since the meeting it had been brought to the Authors' attention that references to Brooklands were made in following publications:

Royal Dawson, "Curve Design." Spon, 1932, p. 171.
"The Brooklands Motor Track Bridge." *Conc. & Constr. Engng*, vol. 2, 1907–8, pp. 240 and 328.

Correspondence on the foregoing Paper is now closed and no contribution other than those already received at the Institution will be accepted.—Sec. I.C.E.

AIRPORT ENGINEERING DIVISION MEETING

12 January, 1954

Mr R. M. Wynne-Edwards, Member, Chairman of the Division, in the Chair

A Lecture entitled "**The Control of Aircraft Movements at Airports**" was delivered by Clifford Heyes, B.Sc., A.M.I.C.E., and E. J. Dickie, M.B.E., and, on the motion of the Chairman, the thanks of the Division were accorded to the Lecturers.

The following is a summary of the Lecture.

Operational

The calls which those responsible for operating and controlling aircraft make on the engineer are many and tend to change frequently to keep pace with development of new aircraft and new equipment.

Among airport aids to the pilot are obstruction lights, approach-runway and taxi-lights, and the I.L.S. and G.C.A. bad-weather-approach systems. The controller needs extensive landline and radio communications, direction finding equipment and, at larger airports, radar to detect aircraft in the air and to scan the airport surface in conditions of bad visibility. A semi-automatic system for ground movement control is under consideration.

The airport control tower houses the traffic control staff, a considerable amount of radio, radar, and lighting control equipment, and sometimes also the meteorological office and departure clearance office. All have specialized requirements for lighting, heating, sound-proofing, ventilation, and air-conditioning, and need various forms of message-handling equipment.

As traffic speeds and density increase an extremely high standard of reliability becomes essential and everything possible has to be done to guard against power-supply or equipment failure.

Engineering

The Approach and Landing

Radio aids are divided into two entirely independent systems. First, the instrument landing system which is essentially radio, and gives the pilot information as to position over the extended centre-line of the runway both in alignment and approximate distance from the threshold, and his angle of descent; secondly, the ground-controlled approach system in which an observer on the ground sees the image of an incoming aircraft

on a cathode ray tube and advises the pilot by radio telephony on his actual position and angle of descent.

The visual type of aid to incoming aircraft consists of lighting systems which commence with the location and identification beacons, the former advising the site of an airport, and the latter flashing code letters. This is followed by the approach and runway lighting systems which are probably the most important lighting aids on the airport.

The functions of approach lighting are to indicate :—

(1) The extended centre-line of the runway.
(2) Distance from the threshold.
(3) A horizon.
(4) Correct angle of descent.

These four functions are combined in the line and cross-bar approach-lighting system which is used at all major airports in Britain ; the centre-line extends to a distance of 3,000 feet from the threshold, and two or three cross-bars are located at 1,000-foot spacing. The length of crossbars is such that all appear to be the same when the aircraft is descending at the correct angle and height, and the attitude of the aircraft is checked by the horizon effect.

The circuits supplying the approach lighting are duplicated, lamps being interleaved between the two circuits to maintain the pattern should trouble develop on one circuit. Brilliancy is controlled to suit visibility conditions, a maximum of 20,000 candelas being desirable. Circuits have been 230-volt parallel, but series circuits are now coming into use, providing equal brightness of all lamps.

Runway lighting should approximate in brilliance to approach lighting. This is not practicable with flush lighting fittings which can be run over by aircraft without risk ; hence, elevated runway lights are coming into use, fixed adjacent to, but just off, the runway edge. They have very small mass, so hazard to aircraft is slight. Intensities up to 20,000 candelas can be obtained, and brilliancy control may be applied. Series circuits are used, the power supply for alternate lamps being taken from opposite ends of the runway.

Obstruction lights are required to mark tall buildings and natural obstructions close to the approaches to an airfield. These are dual lamp fittings with red filters placed at vertical intervals not exceeding 150 feet, whilst the length of the obstruction is delineated together with its highest point. Control is effected by land-line if convenient or the lamps may be underrated and burn continuously.

Movement over the Surface

Taxi-tracks are usually marked with flush blue lights along both edges of the track. On a major airport having multiple taxiways, green centre-line lights may be laid in the track, the pilot keeping astride this line.

Parallel circuits are used for edge lighting, and series circuits for centre-line lighting for reasons of electrical control. In the case of very complicated taxi-tracks, a block system is used to split up the tracks into sections, requiring the provision of red stop-bar lights across the track.

Traffic lights control the movement of vehicles crossing the ends of operational runways, and at some airports traffic lights are required on public roads adjacent to the ends of runways. In both circumstances control of the lights is either from the tower or the runway controller.

The detection of aircraft on the surface in times of bad visibility, other than by radar, is being achieved by an electro-magnetic device consisting of sets of coils in the runway, and indication of an aircraft is obtained by the unbalance set up in the electrical coupling between the oscillator and receiver coils. This system is intended ultimately for automatic operation.

Control of Ground Services

The electrical control system in general use operates by transmitting coded pulses from the control desk to controlled centres throughout the airport, in which relays operate contactors to bring on the power loads represented by radio aids or lighting. Back-indication is provided by a current transformer operating a relay through a bridge rectifier and showing the state of the services on a mimic diagram. In fault conditions a warning light appears on the control desk, and maintenance crews investigate and correct. To remove the burden of reporting faults from the controller, a system of electronic scanning is about to be installed at a large airport to report the state of the system by using cold cathode tubes to form the counting chains, which select the circuits to be tested in orderly routine.

Electricity Supply

Main supplies are invariably taken at high voltage from the local Electricity Board, wherever supplies are possible. Distribution around the airfield is also at high voltage, and ring mains are used for all important services. To provide an emergency supply within 2 minutes of failure, the essential electrical loads at the airport are covered by automatic stand-by sets ranging in size from $2\frac{1}{2}$ kilowatts to 250 kilowatts.

Protection of the network is generally by selective setting of circuit breakers on the ring main and fused isolators to each transformer. On some of the larger airports, balanced feeder protection is applied.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

2 February, 1954

Lieutenant-Col. R. H. Edwards, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Maritime Paper No. 25

"Site Exploration for Maritime and River Works"

by

Michael John Tomlinson, A.M.I.C.E.

SYNOPSIS

An account is given of the procedure in making borings and in-situ tests from floating craft or stagings for site exploration in connexion with the design of maritime civil engineering works. Methods of setting-out, selection of suitable vessels for making borings, and the various methods of boring to suit different soil or rock conditions are described. Stagings are preferred wherever possible, and a diver is often helpful in difficult site conditions. The application of "standard" and cone penetration tests, and vane shear tests to marine site exploration are discussed and data are given on typical soil conditions in estuarine and river deposits. It has been found that the cone penetration test is a very suitable method for investigating the nature and consistency of highly stratified deposits in connexion with the design of deep pier and long friction-pile foundations. Comments are made on driving and loading test piles on off-shore sites. The latter part of the Paper comprises an account of driving test piles in deep water, and of actual site investigations made for oil- and ore-loading piers, and for determining the stability of a sea wall and coastal cliffs.

Introduction

A careful study of the ground conditions is an essential preliminary for all civil engineering works. The main objects in obtaining information on the ground conditions are defined by the Civil Engineering Code of Practice for Site Investigations [1] as follows:—

"(*a*) To assess the general suitability of the site for the proposed works;
(*b*) To enable an adequate and economic design to be prepared;
(*c*) To foresee and provide against difficulties that may arise during construction due to ground and other local conditions;
(*d*) To investigate the occurrence or causes of all natural or created changes of conditions and the results arising therefrom."

[1] The references are given on p. 255.

Although the great engineering works of the nineteenth century were designed from long experience and intuitive judgement, the designs were none the less based on adequate knowledge of the foundation conditions, and reference to trial pits and borings can be found in many historical works. To explore the foundation conditions for his Glasgow Bridge, Telford [2] (1757–1834) sank a cylinder 72 feet below the river-bed, and his plans for the design and construction of the piers were based on the findings. The possibility of a " blow " in the sheet-piled cofferdams was foreseen and plans were made in advance to deal with such an occurrence. Telford's ingenious method of constructing the Clachnacharry sea locks of the Caledonian Canal was based on his understanding of the phenomena of plastic yield and consolidation of soft soils. These locks were constructed by tipping an embankment of boulder clay from the shore to the site of the locks where the subsoil was shown by borings to consist of 50 feet of soft mud. A stack of masonry was placed on top of the boulder-clay embankment so that consolidation of the boulder clay and plastic yield and consolidation of the underlying mud took place under a load equal to or greater than that of the masonry of the lock. After periodical levelling had shown that settlement had ceased, the lock pit was excavated and the masonry walls and invert were constructed.

The present-day advances in the science of soil mechanics have aroused widespread interest in scientific methods of site exploration. The publication of the Code of Practice for Site Investigations [1] has helped to establish the correct procedure in making investigations, and has standardized the description and classification of soil and rock types.

The purpose of this Paper is to describe the special procedure and technique for making site investigations for maritime and river works, and to give some examples of the influence of various soil conditions on design and construction of the works.

It is not proposed to describe methods of boring and soil sampling in detail, since these have been fully described in the Code and elsewhere.[3]

Information to be Obtained from Site Exploration

Assuming that the site (or alternative sites) has been selected from considerations of geographic and economic suitability, then the object of the site investigation is to obtain the undermentioned data.

(*a*) A detailed topographical and submarine survey of the site giving :
- (i) outline and contours of the coastline, foreshore, sea-bed, and other topographical features ;
- (ii) levels of the tides or other fluctuations in water levels ;
- (iii) location and details of existing marine structures ; and
- (iv) location and details of wrecks or other obstructions on the sea-bed.

(*b*) Observations on the rate of tidal fluctuations, velocity and

direction of currents, formation of waves, scour and siltation of the sea- or river-bed, movement of foreshore material by drift, stability conditions of beaches, dunes, cliffs, and training works.

(*c*) An exploration of the surface and subsoil conditions covering the area of the proposed works and for the full depth of subsoil affected by excavations and structural loadings. The exploration is made to determine the nature and sequence of soil or rock strata, the geological structure of the site, and the ground-water conditions. Soil samples are obtained to provide test data to aid the design and construction of foundations to structures and earthworks.

(*d*) Collection of rainfall, storm, wind velocity and direction, fog, temperature, and humidity records.

(*e*) Observations on the condition of existing marine structures, such as attack by marine growths and borers, corrosion of metal work, disintegration of concrete, and attrition by floating debris or sea-bed movements.

(*f*) Investigations of available constructional materials, such as aggregates for concrete production, stone for blockwork, and earth for reclamation.

(*g*) Location of sites for disposal of dredged or other waste materials.

(*h*) Information on the requirements of statutory bodies, such as port authorities, conservancy and river boards, local planning authorities, and fisheries.

The scope of this Paper is mainly confined to a description of the procedure for obtaining information under the heading (*c*).

Boring and Sampling

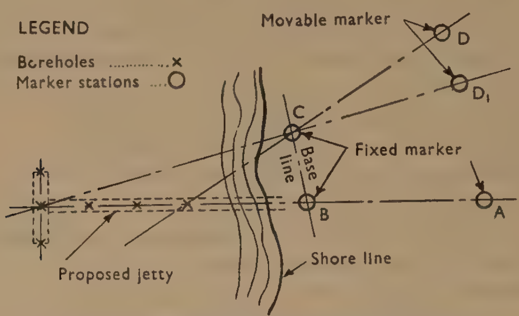
Lay-out and Location of Borings

A tentative lay-out of borehole positions is marked on a survey map of the area under investigation. This map should show the sea-bed contours; any areas of marked irregularity in contour spacing which suggest deep erosion channels or submerged rock pinnacles should be specially investigated in the boring programme. Survey stations are then established on shore, from which the boring craft is located over the desired positions.

Methods of locating the craft from the survey stations depend on the purpose of the investigation. If a large area is to be explored by many shallow "probings" (uncased jetted borings), it is satisfactory to work along a ranging line, using targets on the shore. The probings are made at roughly equal distances along the ranging line, the position of the craft at each probing being established by sextant observations. Shell and auger borings made to investigate the subsoil at specific points on a proposed

structure—for example, on the site of each pier of a multi-span bridge—are located by the procedure shown in *Fig. 1.* With well-sited and clearly visible shore markers, and freedom from strong winds, heavy swell, or currents, the boring craft can be located and held in position with surprising accuracy. Secure positioning of the craft is an essential condition for successful marine boring, and is often the most difficult feature of the whole operation.

Fig. 1



LOCATING BOREHOLES FROM SHORE MARKERS

The lay-out of anchors and alignment of the craft depend on the direction of winds and currents, and the "holding" of the sea-bed for the anchors. In an exposed situation with strong tidal currents, there should be at least six anchors—one at the bow, one at the stern, and one at each quarter. In sheltered waters one bow and two stern anchors may be all that are needed. The crews on the winches should be well-drilled and alert, since constant adjustment is needed to the length of the hawsers on rising and falling tides. A separate winch is required for each anchor, otherwise a multi-drum winch should be provided. Power-driven winches speed up the operations of picking up moorings and re-positioning the vessel, and are valuable if frequent moves have to be made. The heavier the anchor, the more accurate is the control of position. A 2-cwt anchor supplemented by a short length of heavy anchor chain is a minimum weight for holding a lighter in the open sea. Anchors of this size are laid out by a tug or motor launch.

Boring Craft

The craft used for boring should have adequate seaworthiness to ride out a moderate storm when anchored in position. It is inconvenient and hindering to progress to have to pick up the moorings and seek shelter in a harbour every time a gale or squall threatens. Ample deck space is necessary to lay out the boring equipment and mooring winches, and the deck construction should be strong enough to carry the cantilevered boring platform, and to hold down the winches.

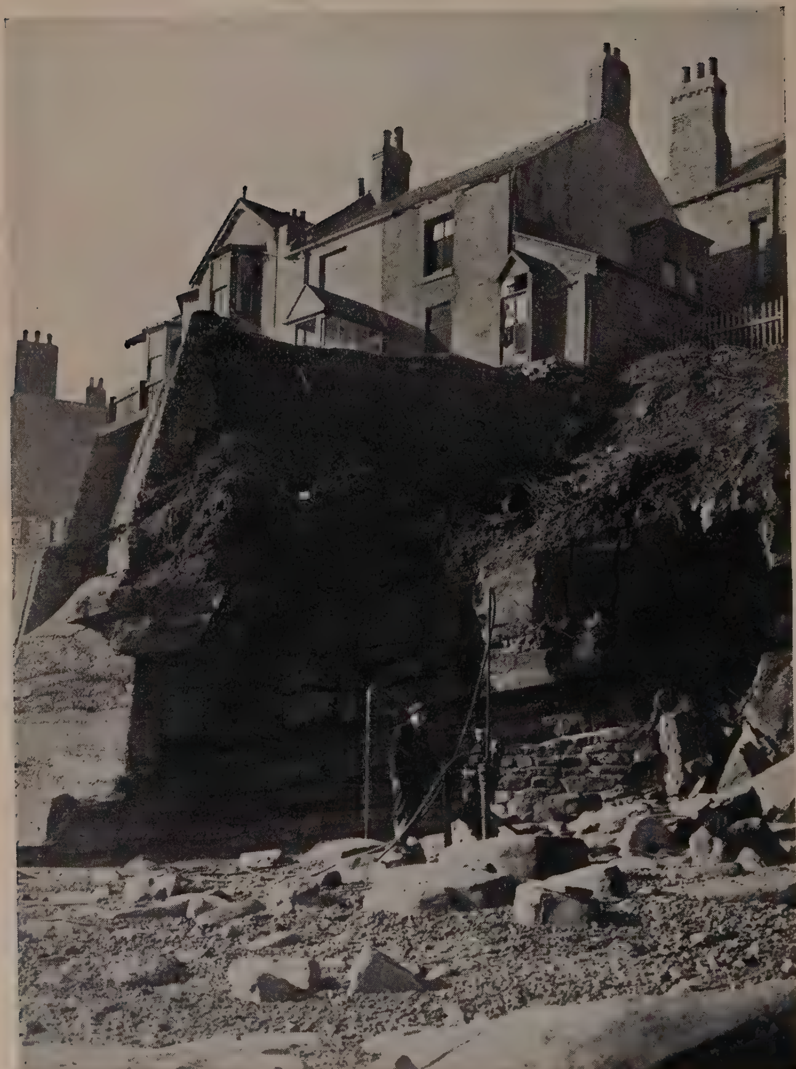
Fig. 3

Boring Rig mounted on Ore Lighter for Off-Shore Boring in Cyprus

Fig. 4

"Beetle" Steel Cell Pontoons used for Borings at Little Aden

Fig. 20



DRILLING AT FOOT OF FALLEN CLIFFS AT CULLERCOATS, NORTHUMBERLAND
(Note erosion of soft strata beneath more resistant sandstone bands.)

The craft should be of sufficient size to give a stable working platform in choppy seas, and should be sufficiently manœuvrable to enable it to be towed into position by tugs or launches—a self-propelled vessel is not necessary.

To meet the above requirements a rectangular pontoon made up from watertight steel-plate cells is, in the Author's opinion, the most suitable type of craft. Such vessels are very seaworthy, they can be readily ballasted, and if swim ends are provided they are sufficiently manœuvrable for most conditions. The lay-out of a boring pontoon is shown in *Fig. 2*. The decision whether to mount the boring platform on the bow, stern, or amidships, depends on the available deck space, the need to moor the craft in a direction prescribed by currents or navigation restrictions, and the type of boring rig. The rig shown in *Fig. 2* is the mast-type, which has advantages over the tripod or four-legged derrick for marine work. With the latter type, there is a risk of trapping the borehole casing inside the bracing of the legs of the rig when the tide is falling swiftly.

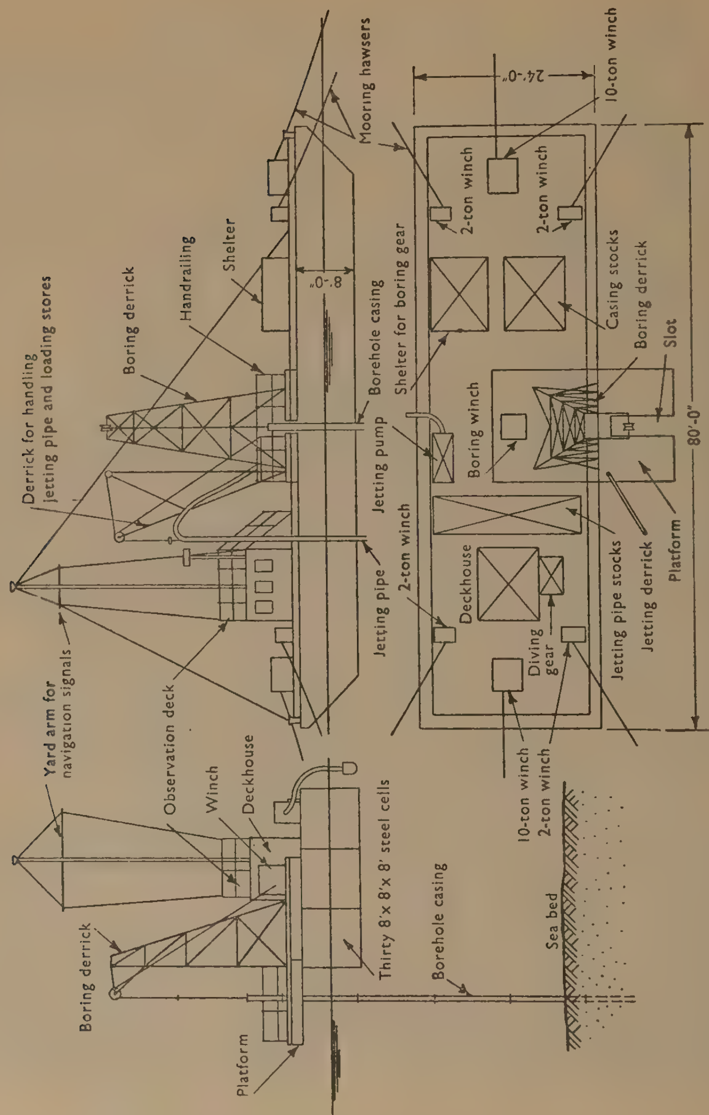
It is not common practice in Great Britain to use spudded boring vessels, but they are frequently used in the United States of America, in a similar manner to spudded suction dredgers. The spuds consist of massive timber or steel vertical columns passing through guides at the corner of the pontoon. Their lower ends rest on the sea bed, and by hauling on block and tackle gear, the pontoon is raised up the spuds until a state of "negative buoyancy" is reached, when movement of the pontoon by waves is eliminated or reduced to a small amplitude. The pierhead pontoons to the Mulberry harbour were a notable example of the use of spuds.

Spudded craft are essential for large-scale rotary blast-hole drilling in open water; such work cannot be done in any other way, except perhaps from movable stagings resting on the sea-bed. Where blast holes are closely spaced, large pontoons are used with row of drills along each side.

An ingenious adaptation of the spudding method was used by an American boring contractor for sinking 250-foot-deep holes at distances up to 10 miles off-shore in Lake Maracaibo, Venezuela. A triangular floating platform was fabricated from 22-inch-diameter steel pipe, and 120-foot-long spuds were dropped on to the sea-bed through vertical tubular guides incorporated in the platform. Offshore oil-well drilling platforms have also been constructed in America by lowering tubular supports on to the sea-bed through guide holes in a floating cell pontoon. Special jacks are then used to raise the pontoon up the supports above high-water level.

Generally, the high cost of constructing and maintaining special boring craft, and the cost of transporting them from job to job, is prohibitive, and it is therefore usual to employ any suitable local vessel which can be adapted to carry a boring platform and mooring winches. Motor launches

Fig. 2



General Arrangement of Boring Pontoon

or fishing drifters are suitable for making shallow probings or wash borings if they have sufficient clear deck space. Sea-going lighters are suitable for deeper and cased borings, since the large hatches can be decked over to accommodate the equipment. An ore lighter used for borings off the coast of Cyprus is shown in *Fig. 3* (facing p. 228). Landing craft have been successfully used, and their shallow draught is helpful when working close inshore. Salvage vessels are often too encumbered with capstan winches, cranes, and other lifting gear to enable them to be adapted for rig boring, but they have been used for making probings.

Two fishing drifters lashed together were used on a very extensive programme of probings and wash borings made for the Argyll County Council, in connexion with the reconstruction of harbours and ferry piers in the western islands of Scotland.

The most successful improvised craft so far used was a raft made at the suggestion of Sir Bruce White, Wolfe Barry & Partners, by coupling together two of the "Beetle" pontoons which were used to carry the floating roadway of the Mulberry harbour.[4] These pontoons consisted of watertight steel cells bolted together by projecting flanges. Steel-plate diaphragms were made to couple the two pontoons together and a timber deck was prefabricated in sections to carry the boring rig and winches. The pontoons were shipped in the form of separate cells, and were assembled on site to form two boring vessels which were used on the survey for the new oil port at Little Aden (*Fig. 4*, facing p. 228).

The Use of Staging

Wherever possible, fixed staging is used in preference to floating craft, since the former allows boring to continue in sea conditions unfavourable for floating craft. Time saved by being able to continue boring in all weathers and at all states of the tide, when working close inshore, may more than offset the high initial cost of constructing a long or high staging. However, there are circumstances which prohibit the use of staging, where for instance it will obstruct navigation channels, or on sites where there is a very high tidal range (in the Severn Estuary the tidal range is as much as 50 feet). Staging can be used at long distances from the shore in the form of prefabricated trestles lowered on to the sea-bed by a crane barge.

When using staging in rivers, the height of the staging should allow for normal flood flows, and precautions should be taken against damage by floating debris or ice. Tubular steel scaffolding has been found the most suitable material. It can be easily handled without cranes, and a properly designed and well-braced scaffold can withstand heavy seas.

Fig. 5 (between pp. 228 and 229) shows a tubular steel scaffold staging used for rock drilling below the bed of the River Severn at Shrewsbury, in connexion with a scheme to construct a sewer in tunnel beneath the river. The borings were made in the winter, so the boring platform was placed

above the normal winter flood level, and the staging was well tied back to the banks to preven overturning as a result of the accumulation of debris on the upstream side.

The Use of a Diver

Although a diver is an asset to any marine boring operation—for recovering tools lost overboard, raising anchors cut away in emergency, and raising and re-connecting casing broken off below water level—the cost of his continuously standing by is not usually justifiable. In some circumstances a diver is essential. For example, there may be hard rock on the sea-bed preventing the casing from being pitched and driven. The diver can lay explosive charges to blast a hole in the sea-bed, and can then guide a weighted chisel operated from the derrick to enlarge the blast hole for further charges, aided by water jetting to wash away the fragments. He also performs a useful service in reporting on the general sea-bed conditions, such as the presence of fissures, ledges, or pinnacles in a rocky sea-bed. A diver can help to speed up progress in bad weather. In an investigation off the Syrian coast, boring work was carried on through two winter seasons using a 350-ton Arab schooner as the boring vessel. The sea conditions were often too rough to permit the use of borehole casing in the conventional way through the slot in the cantilevered platform. Instead, after pitching and driving the casing from the deck of the schooner, the casing was unscrewed just above the sea-bed. The diver then guided the boring tools and sampling equipment into the projecting end of the casing as shown in *Fig. 6*.

A diver is necessary for other aspects of marine investigations, such as surveying the sea-bed conditions before constructing submerged pipelines, or for examining the condition of existing structures below the water line. In the course of time this work may be done by the recently developed under-water television camera.

Methods of Boring and Sampling

The various methods used for boring over water are no different from those used for land borings. Methods commonly used are :

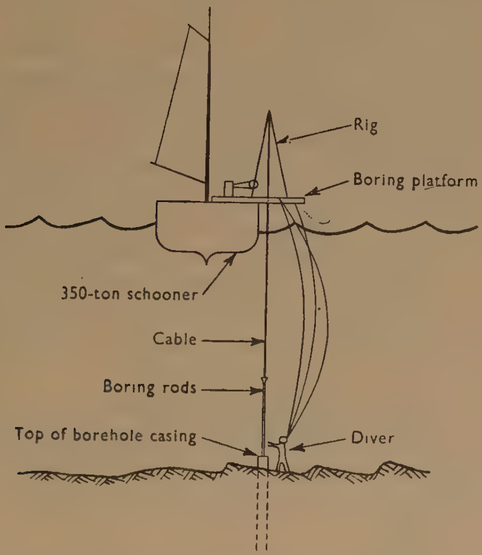
(*a*) probing ;
(*b*) wash boring ;
(*c*) shell and auger boring ;
(*d*) percussion boring ;
(*e*) rotary core drilling.

Probings

The Author has made a clear distinction between probings and wash borings, since they differ in several important details. Probing consists of lowering a small-diameter pipe into the soil and washing away the soil ahead of the pipe with a high-pressure jet until a hard layer is reached

which prevents further penetration. Since the wash-water escapes at sea-bed level, there is no way of identifying the soil passed through, other than by the "feel" of the jet pipe as it is worked up and down, and by observations of wash-water pressure. Soil samples can be obtained by means of a small-diameter tube lowered through the jet pipe, but if water

Fig. 6



BORING IN ROUGH SEAS WITH AID OF DIVER

under high pressure has been used, the samples may be completely misleading. In spite of these drawbacks, probings are often a rapid means of filling in details of the levels of well-defined strata between widely spaced cased borings. They are particularly suitable for the expeditions investigation of large areas for dredging schemes.

Wash Boring

The procedure for wash boring consists of working down a 1-inch-diameter wash pipe, in conjunction with a $2\frac{1}{2}$-inch-diameter casing. A comparatively low pressure and small quantity of water is needed to raise the debris through the small annular space between the wash pipe and casing, therefore there is little risk of compaction of loose sands, or disturbance of soft clays below the bottom of the borehole. The wash-water is discharged into a recovery tank where the cuttings can be continuously examined. Compact and cemented sands or stiff clays are loosened by the

chopping bit, but wash boring is unsuitable in large gravel, cobbles, or boulders. "Dry" samples are taken at closely spaced intervals or whenever a change is observed in the colour or consistency of the soil. These samples are taken with the standard 2-inch-outside-diameter split-spoon sampler, the use of which enables the relative density of cohesionless soils or the consistency of cohesive soils to be determined in situ with sufficient accuracy for most site investigational work, particularly if correlated with the results of tests on undisturbed samples from a relatively small number of larger-diameter boreholes.

Although widely employed in the United States and described as standard practice by Terzaghi and Peck,[5] wash-boring methods have tended to fall into disfavour in Great Britain. The Author feels that the comments given in paragraph F.124 of the Code of Practice for Site Investigations [1] do not give a fair indication of the potential accuracy and reliability of the method when it is conscientiously carried out by experienced operators. As previously stated, the wash-water pressure is low, and the compaction given to loose sands is probably much less than that caused by "shelling," as practised in shell and auger borings. In very soft clays, shells must be used to raise the material to the surface. This causes a disturbance and loss of shear strength, which does not occur when wash boring with low-pressure water.

Ground-water conditions cannot, of course, be reliably assessed from wash borings. Shell and auger boring must be used for accurate investigation of the extent of, and artesian pressure conditions in, water-bearing soil or rock strata.

Shell and Auger Boring

The method of shell and auger boring, described in paragraph F.122 of the Code of Practice for Site Investigations, is universally used for boring in soils or soft rocks where 4-inch-diameter undisturbed samples are required. The boring procedure for marine work is exactly similar to that used for land work, but greater care is necessary in handling the tools and in coupling-up casing and rods. Great care has to be taken to maintain the verticality of the casing in the early stages of boring, and it is frequently necessary to wait for periods of slack water before pitching and driving the first lengths of casing. Difficulty can arise when extracting the casing. A sustained pull on the boring winch is a risky operation since rolling of the vessel can put a severe strain on the derrick structure. Powerful jacks are useless since, when operated from cantilevered staging, they merely cause the craft to heel over instead of raising the casing. By lashing the casing to the vessel, a tidal lift can be used to draw the casing, or pumping from ballast tanks will give the same effect. Difficulties in casing extraction can be minimized by progressive reductions in diameter, and by using either flush-coupled or collared casing, depending on the type of soil.

Percussion Boring

As stated in the Code of Practice (F.123), percussion boring methods, using repeated blows of a cable-operated bit or chisel to loosen the ground, are unsuitable for site investigational boring.

Rotary Core-Drilling

Rotary drills with devices for recovering rock cores are used for drilling deep exploratory boreholes in hard rock formations. Diamond core-drills, in which the drill is advanced by screw or hydraulic feed, are unsuitable for operations on floating craft, except in very calm waters or where spuds are used. Stagings are needed for such drills, but light portable drills can be mounted on casing driven into or supported on the sea bed and guyed to anchorages. An interesting account of the latter method for boring in the Severn Estuary has been given by Symington.[6] Rotary drilling using shot or diamond core-drills in which the drill rods pass through a rotating kelly or table can be used for boring from floating craft, provided that the vertical movement of the boring platform is not more than 1 to 2 feet. Using the rig shown in *Fig. 7* (between pp. 228 and 229), cores have been extracted from a depth of 35 feet below the sea bottom in 33 feet of water.

In-Situ Methods of Soil Testing

It is often advantageous to obtain the shear strength, density, and other soil characteristics by in-situ tests instead of by laboratory tests on undisturbed samples. Time is saved by using these methods, and the cost of transporting samples to the laboratory is eliminated for all but a few samples from boreholes sunk to correlate the in-situ tests. This is particularly advantageous on overseas work. It is difficult to extract satisfactory samples of cohesionless soils from boreholes, and the time required to use special devices, such as the compressed-air sampler, may prohibit the use of these devices for boring from floating craft in open water.

Methods used for making in-situ tests on marine investigations are :

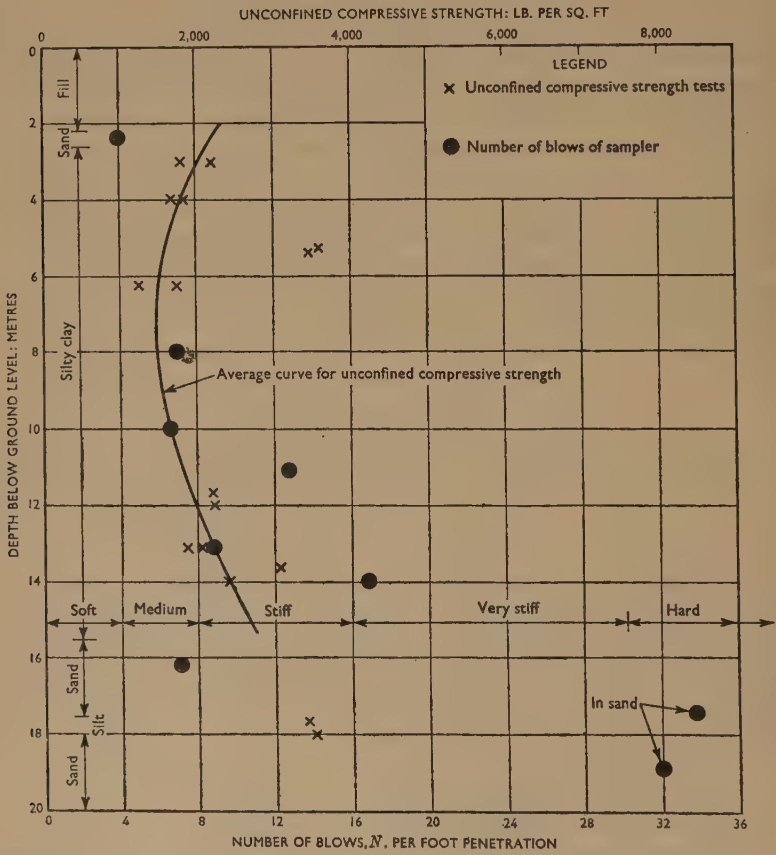
(*a*) the standard penetration test ;
(*b*) the cone penetration test ; and
(*c*) the vane shear test.

The *standard penetration test* was developed in the United States for use with wash borings, and is made by driving a " split-spoon sampler " of standard dimensions into the soil at the bottom of the borehole. The number of blows of a falling weight required to drive the sampler a prescribed distance is recorded. The sampler, which has an outside diameter of 2 inches, is shown in *Fig. 8* (between pp. 228 and 229). The empirical relationship between the number of blows and the unconfined compressive strength of cohesive soils, and the relative density and angle of internal

friction of cohesionless soils, has been given by Peck, Hanson, and Thornburn.[7]

A correlation between the standard penetration test and the measured unconfined compressive strength of the alluvial silts and clays of the River Tigris at Hindiya is shown in *Fig. 9*. The results on this site showed

Fig. 9



RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND NUMBER OF BLOWS, N, IN STANDARD PENETRATION TEST IN ALLUVIAL SILT AND CLAY OF RIVER EUPHRATES AT HINDIYA

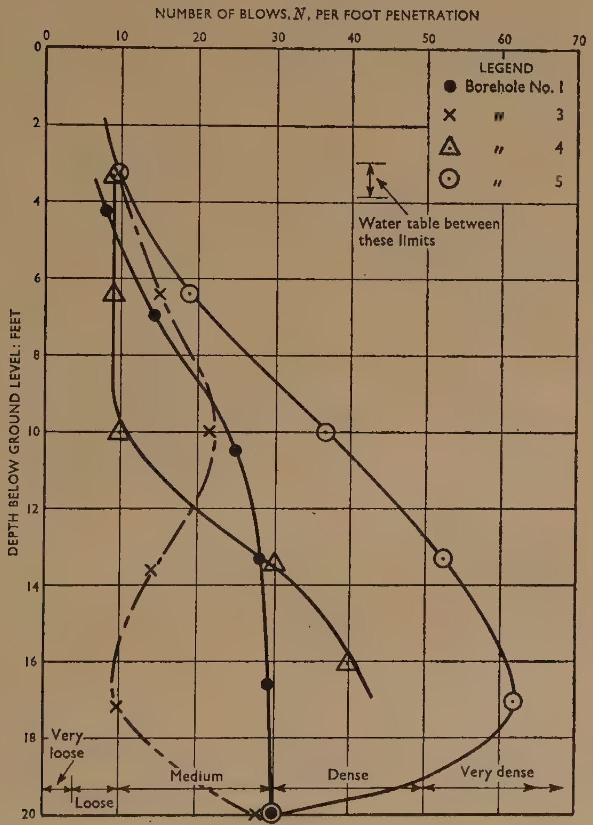
good agreement with the correlation data of Peck, Hanson, and Thornburn. Standard penetration tests made in a number of boreholes in the raised beach deposits of Morfa Harlech are shown in *Fig. 10*.

The test is very suitable for use in conjunction with wash borings

from floating craft. It is simple and relatively quick to operate and gives reliable and reproduceable results.

The *cone penetration test* was developed on the Continent, but has been the subject of increased interest by engineers in Great Britain and the United States. The test consists of forcing a conical point of standard dimensions into the soil by direct thrust or by blows from a falling weight.

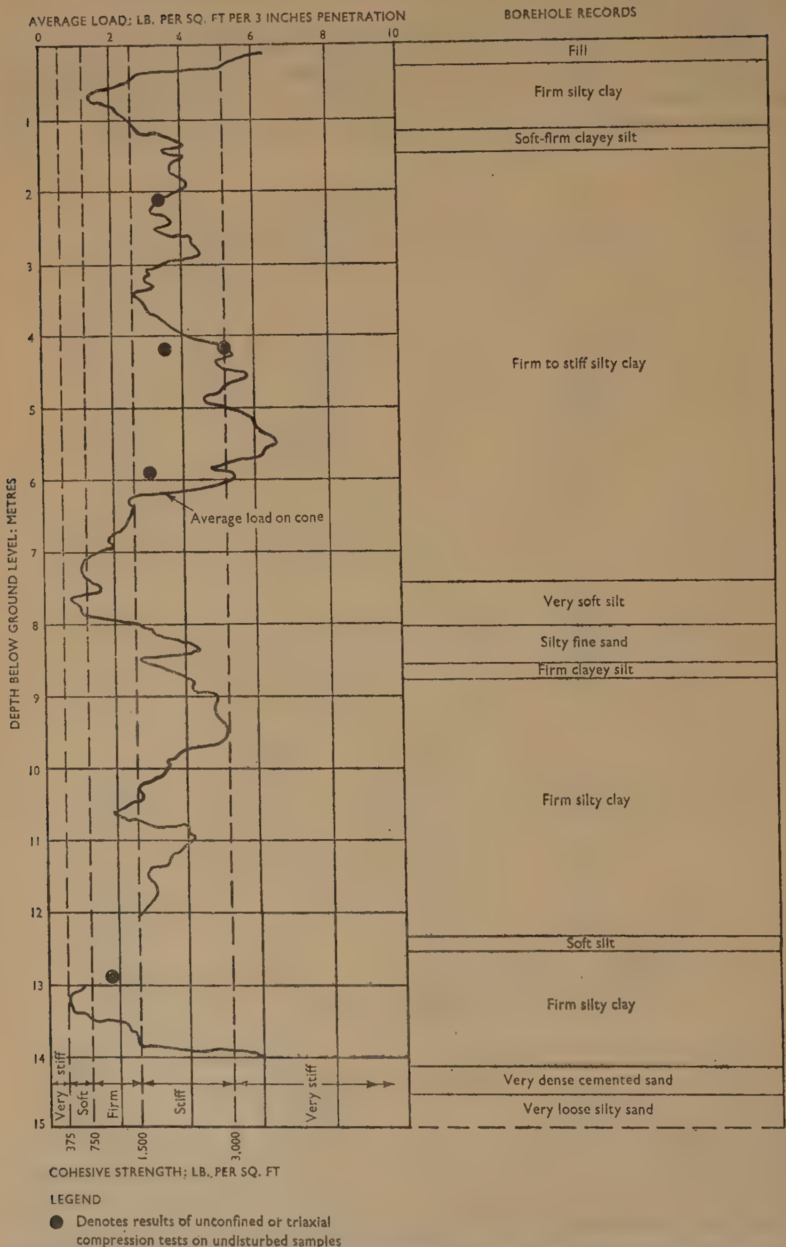
Fig. 10



STANDARD PENETRATION TESTS IN SANDS OF RAISED BEACH AT MORFA HARLECH, NORTH WALES

These methods are called the " static " and " dynamic " tests respectively. The static test is more generally used and its application to the design of piled foundations has been described by Geuze [8] and Huizinga.[9] Because of the fixed resistance which is needed for jacking down the cone, the static test has limitations for use on floating craft, except in very calm waters. The test is generally made from fixed staging. Another drawback

Fig. 11



Correlation of Static Cone Penetrometer Test with Laboratory Tests in Alluvial Silts and Clays of River Euphrates at Samawa

is its lack of sensitivity in very soft soils. This was experienced in tests made from stagings in 40 feet of water for a submarine drilling platform in North Borneo. The cone showed a relatively high resistance through 5 feet of sand on the sea-bed, but after passing through this layer the cone dropped 50 feet under the weight of the rods and casing and no resistance could be recorded. A record of a static cone penetration test compared with shear strength tests is shown in *Fig. 11.* These results were obtained in the alluvial sands, silts, and clays of the River Euphrates at Samawa.

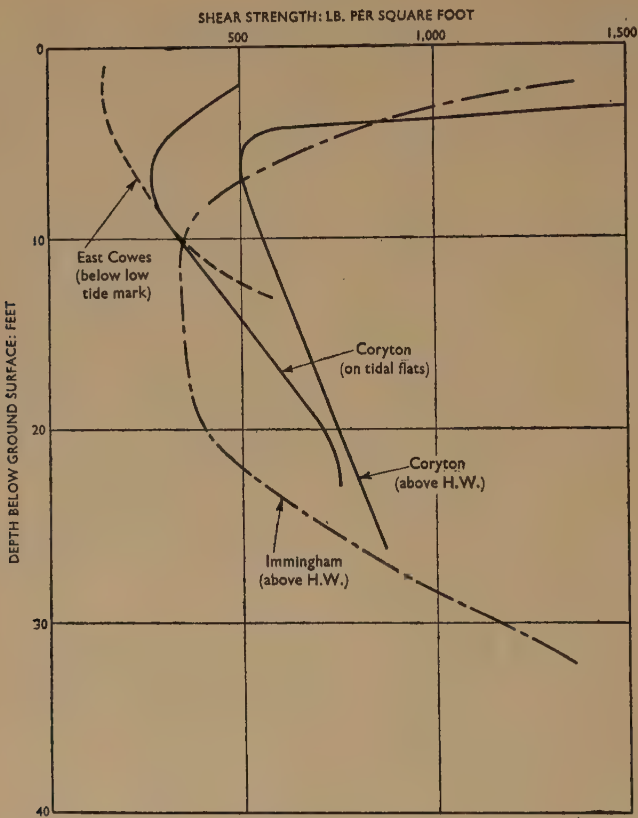
The sequence of strata and the variation of shear or unconfined compressive strength with depth shown in *Figs 9* and *11* are typical of the alluvial deposits of the middle reaches of the Tigris and Euphrates on a number of sites investigated by the Author's firm. Heavy structures, such as bridges, constructed in these soil conditions are usually founded on deep caisson piers or long friction piles. The cone penetration test gives an accurate and detailed picture of the nature and extent of the alternating sand, silt, and clay layers, enabling the skin friction on piers and piles to be assessed with some accuracy. With normal boring and sampling methods, continuous undisturbed sampling would be necessary to obtain this information. However, the cone penetration test does not eliminate the need for boring, since samples must be obtained to describe and classify the soil, and the ground-water conditions must be fully investigated before constructing deep pier foundations.

The *vane shear test* has been successfully used in soft and sensitive marine and estuarine sediments to obtain an accurate and continuous record of the shear strength of these soils in conditions where conventional methods of sampling and testing would have resulted in disturbance and loss of shear strength of the samples.

The vane test has its limitations for use from floating craft, in a similar way to the cone penetration test, since a fixed reaction must be provided to the torque on the vane rods. Although this reaction can be given by operating the apparatus inside borehole casing, the process of sinking casing, and movement of the casing owing to oscillation of the boring platform, leads to loss in shear strength for several feet around and below the casing, in very soft sensitive soils. For this reason, when determining the shear strength of the soil at shallow depths below the surface, the vane test should, wherever possible, be made by pushing down the vane independently from borings, with the torque reaction provided by a fixed platform with its supports at least 5 feet away from the test position.

The apparatus used by the Author's firm was developed from that originally used by Evans,[10] and results obtained in estuarine silty clays of the Thames at Coryton, the Humber at Immingham, and the Medina at Cowes, Isle of Wight, are shown in *Fig. 12.* The stiffer surface crust caused by desiccation is very marked for the tests made above high-water mark ; it is less marked for the tests on tidal flats submerged only at high water, and no crust was recorded for tests in fully submerged soil.

Fig. 12



SHEAR-STRENGTH/DEPTH RELATIONSHIP FOR ESTUARINE SILTY CLAYS AT VARIOUS LOCATIONS

GEOPHYSICAL EXPLORATION

Geophysical methods of site exploration are not used to any great extent in maritime civil engineering investigations, but they have been extensively used for submarine oil-prospecting, and for research work into the depth and character of marine sediments in very deep waters. Professor Hans Pettersson, in his account of the Swedish deep-sea expedition in 1947, has described how echoes were recorded by seismic apparatus from reflecting surfaces 1,000 to 3,000 feet below the sea bed in 2,000 to 3,000 fathoms of water. Magnetometer surveys have been made for prospecting mineral deposits below the sea, with the magnetometer trailed behind a ship or aircraft.

Electrical resistivity or seismic refraction methods are the two most suitable for civil engineering investigations. For marine or river work the procedure for electrical resistivity surveying is exactly similar to the procedure on land, except that the electrodes are suspended in the water from floats or a catenary cable instead of being driven into the soil. Used in this way, the water itself behaves as a stratum, which can lead to difficulty in interpretation, particularly in deep and saline water.

Seismic methods are more suitable for use in steep terrain where electrical resistivity surveys are unsuitable. When making seismic surveys over water, the geophones, or vibration pick-ups, are suspended from floats and towed along by a ship, or they may be suspended from tripods placed along the sea bottom. An example of the latter method used in the sub-surface exploration for the St Lawrence River Project is given by Shepard and Haines.[11] In recent applications of seismic surveys to submarine investigations, the geophones have been placed inside containers fitted with double gimbals designed to keep the geophones in a vertical position. A complete string of containers is towed along the sea bottom by a ship, enabling very rapid traverses to be made. In shallow waters the geophones can be lowered into vertical pipes jetted or driven into the sea-bed.

Sonic sounding apparatus [12, 13] is being developed at the present time for sub-surface exploration below water and this may prove to be a valuable method for such work.

Test Piling

Where soil stratification is simple and well defined, the driving resistance and carrying capacity of piles can be predicted, with a fair degree of accuracy, from the results of borings, in-situ penetration tests, and shear strength tests on soil samples.

Previous experience in similar soil conditions can be used as a guide to driving resistance, and static or dynamic cone penetration tests can often be directly correlated to pile-driving resistance. The carrying capacity of piles can be estimated from experience in similar soil conditions, or by calculations based on the shear strength and angle of internal friction of the soils by the methods described by Meyerhof.[14]

However, where there is variable soil stratification, such as in glacial deposits, or where layers of cemented sand or rock are interbedded with sands and clays, the driving resistance and carrying capacity of piles can be determined with accuracy only by driving and test-loading full-scale piles. The cost of such work is not excessive when the piles are driven close to a jetty or river bank, but the work is expensive if piling barges have to be used in unsheltered waters. Test piling in deep waters is also complicated by the need to drive additional piles to support an observation platform for measuring driving resistance, and extensive piled supports

are needed to carry load-testing equipment. The cost of test-loading piles in deep water is, in most cases, prohibitive, but pull-out tests, using the buoyancy of a pontoon on a rising tide, can give valuable data on skin friction. For reconstruction schemes of existing jetties or wharfs, test piles can be conveniently loaded by jacking against a beam anchored to the structure.

In a recent investigation, the stability of a group of piles was determined by observing the strains in each pile of the group when a load was placed on the pile cap. The strains were measured by a direct-reading strain gauge clamped to each pile. The gauge consisted of a 5-foot-long rod with a diamond point on its lower end. Scratches were made on a polished steel plate fixed to the pile for zero load and full load on the cap. The distance between the scratches was measured by an optical device, and a shortening of 0·0005 to 0·0060 inch was measured over the 5-foot gauge length. Comparison of the strain measurements in each pile showed that two of the piles were each carrying about 45 per cent of the total load and the other two about 5 per cent each. It was concluded that there was structural continuity in the greater part of the length of the piles, and that two of them were not broken through completely just beneath the river-bed, as had been feared from the relative inclination of the piles in the group.

Extraction of test piles driven in water often causes difficulty, and most harbour authorities will not permit piles to be cut off at sea-bed level, particularly in areas where future dredging is to take place. Extraction by utilizing the buoyancy of a vessel is generally the only practicable method in deep water, but if a heavy lifting effort is needed, the pile may fail in tension.

Steel H-beam or steel box-piles are useful sections for test piles. They are easily handled without breakage, they stand up well to hard driving, and can be quickly and cheaply lengthened by fish-plating and welding on additional lengths. Heads of steel piles buckled by hard driving can be readily cut down for re-driving.

The plant and floating craft required for test piling are no different from that used for normal piling construction on the water. Information on the procedure for recording the driving and test loading of piles is given in the Civil Engineering Draft Code of Practice for Foundations, Part IV, Sections 1.05 and 1.06, and Appendices A to C.

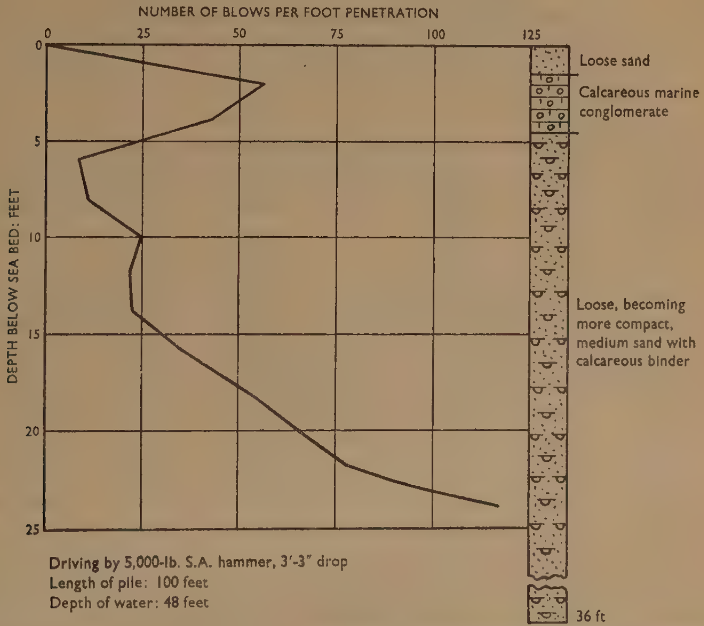
Examples of Site Investigations and the Effect of the Results on the Design and Construction of the Works

Oil and Cargo Jetties at Mina-al-Ahmadi, Persian Gulf

The oil loading and cargo handling jetties at Mina-al-Ahmadi were constructed for the Kuwait Oil Company in 1948–9. An account of the design and construction of this project was given by McGowan, Harvey,

and Lowdon.[15] In view of the magnitude of the work it was considered essential to explore the sub-surface conditions in advance of driving the permanent piles. The first site investigation, which had been made in the winter of 1947–8, had comprised driving trial piles and making borings. The five trial piles of tubular steel section had been driven from a barge along the location of the proposed jetty head, and three of them had

Fig. 13



DRIVING RESISTANCE OF 14-INCH-BY-14½-INCH-BY-73 LB. STEEL H-PILES COMPARED TO RECORD OF ADJACENT BOREHOLE

refused at a negligible penetration; hence, thorough exploration of the depth and extent of the hard stratum on the sea bed was an urgent necessity. Time did not permit the boring equipment being sent out by sea, so the equipment, which weighted 7½ tons in all, was despatched by chartered aircraft and assembled in Shuwaikh Harbour. The borings were made from a 160-foot barge, of 25-foot beam and 9-foot draught, with the boring rig mounted on a platform cantilevered from the side amidships. Probings and a wash boring were made from the barge, and probings in the boat-harbour area, to investigate conditions for dredging, were also made from a 60-foot-long " boom " (a Persian Gulf barge). The jetty head was sited nearly 1 mile from the shore on an exposed part of the coast, and the work was continually being held up by winter gales.

A typical borehole record is shown in *Fig. 13*. The marine conglomerate

at the surface was not continuous over the whole area of the approach and jetties; it was not present near the shore, nor in the area of deepest water at the northern end of the oil loading berths. The conglomerate, which had a maximum thickness of 3 feet 6 inches, was penetrated in the borings by one, or a combination, of the following methods :

(*a*) Heavy chiselling with the chisel guided by a diver until a socket was made into which the borehole casing was placed.

(*b*) Rotary drilling using a diamond core barrel operated by a rotary table mounted on the boring platform.

(*c*) Blasting, with the explosive placed in drill holes or in fissures in the rock.

A diver was employed and was invaluable for examining the rock surface for fissures through which probings could be made, and for setting explosive charges. A charge weight of 3 lb. was generally used ; up to three successive charges were needed to break through the thickest layers of rock.

The results of the borings were subsequently confirmed by the experiences in driving the permanent 14-inch-by-14½-inch-by-73 lb. steel H-piles. Some damage to the piles was caused when driving through the conglomerate, but additional strengthening of the pile toe was found efficaceous for penetrating the thickest layers of rock. A typical driving record is shown in *Fig. 13*. The piles, which had a maximum length of 105 feet, were driven by a 70-foot-high pile frame mounted on a 100-foot-by-29-foot-by-8-foot-draught multi-cell steel pontoon. A single-acting air-driven hammer with a 5,000 lb. ram falling 3 feet 3 inches was used for pile driving. The loose nature of the sand below the conglomerate found in the borings was reflected by the low driving resistance of the piles once they had broken through the rock. This may have facilitated the breaking up of the rock by the piles, owing to the " give " of the sand under the repeated blows on the rock.

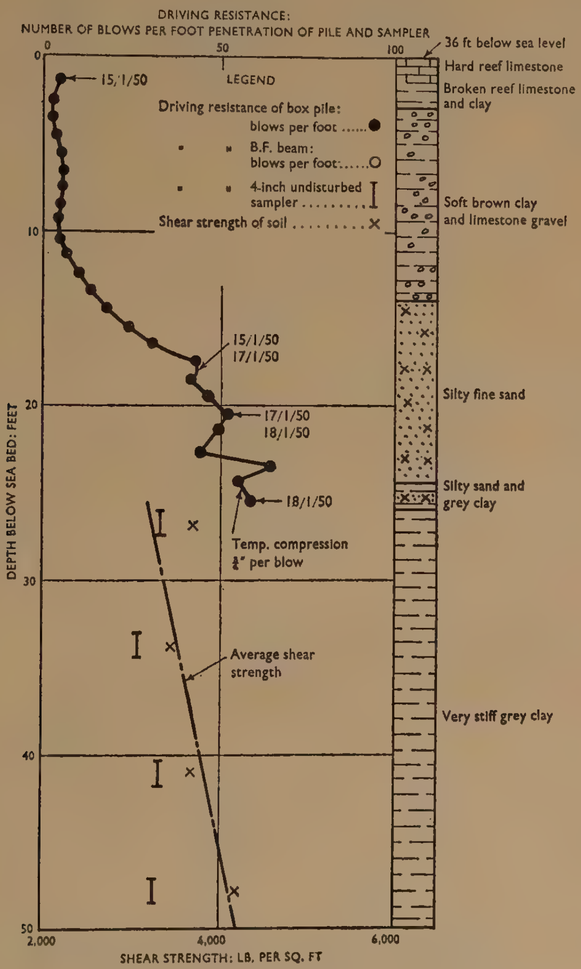
Test Piling for Harbour Project, Syria

A series of investigations was made in 1948–50 for a projected large deep-water oil harbour on the Syrian coast. More than 130 boreholes were sunk below the sea-bed to explore dredging conditions and for jetty and breakwater construction. Diamond-drill borings were made on land on possible quarry sites for the rubble breakwater. Five test piles were driven 1,500 yards offshore in 30 to 40 feet of water to a penetration of 30 feet below sea bed. A tank landing craft, 187 feet by 39 feet by 8 feet, was adapted to mount the 65-foot-high pile frame as shown in *Fig. 14* (between pp. 228 and 229). Other items of plant included :—

1 set of 10-foot-long extension leaders below the pile frame
1 2-ton single-acting hammer
1 3-ton drop-hammer

1 McKiernan Terry No. 7 Extractor Hammer and extracting gear
1 double-barrel piling hoist powered by Diesel engine
1 500-cubic-foot-per-minute air compressor
1 10-foot-by-4-foot air receiver.

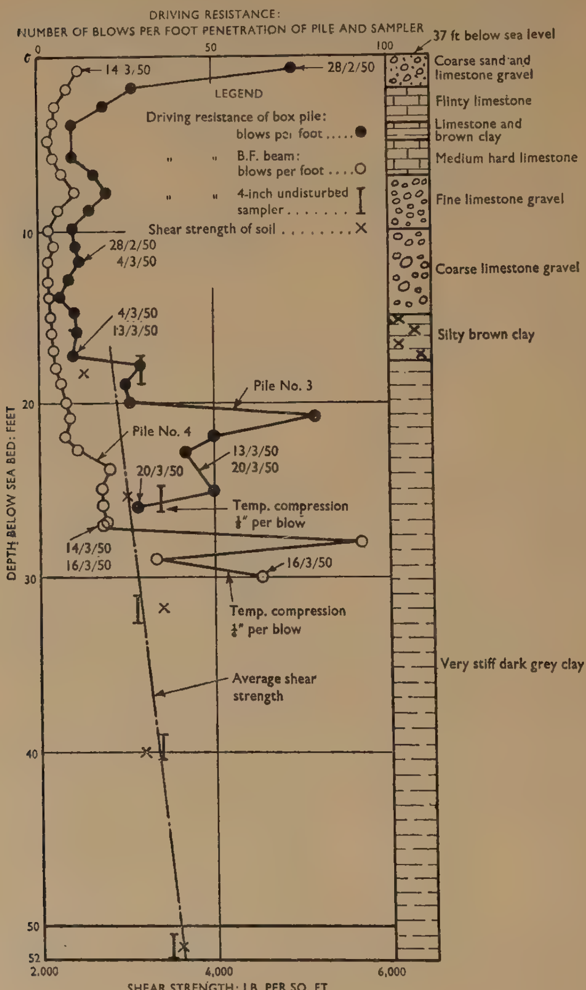
Fig. 15 (*a*)



TEST PILE NO. 1

Note.—Sampler driven by 12-inch blows of 275-lb. hammer for piles 1, 3, and 4, and by 76-lb. hammer for pile 5. Average weight of rods and tube: 250–400 lb. Larssen B.P.3 box pile (closed end full of water). Length: 65 feet. Weight: 2·5 tons. Total penetration: 25 feet 3¾ inches. Hammer: 3-ton drop-hammer falling 3 feet. Timber dolly.

Fig. 15 (b)



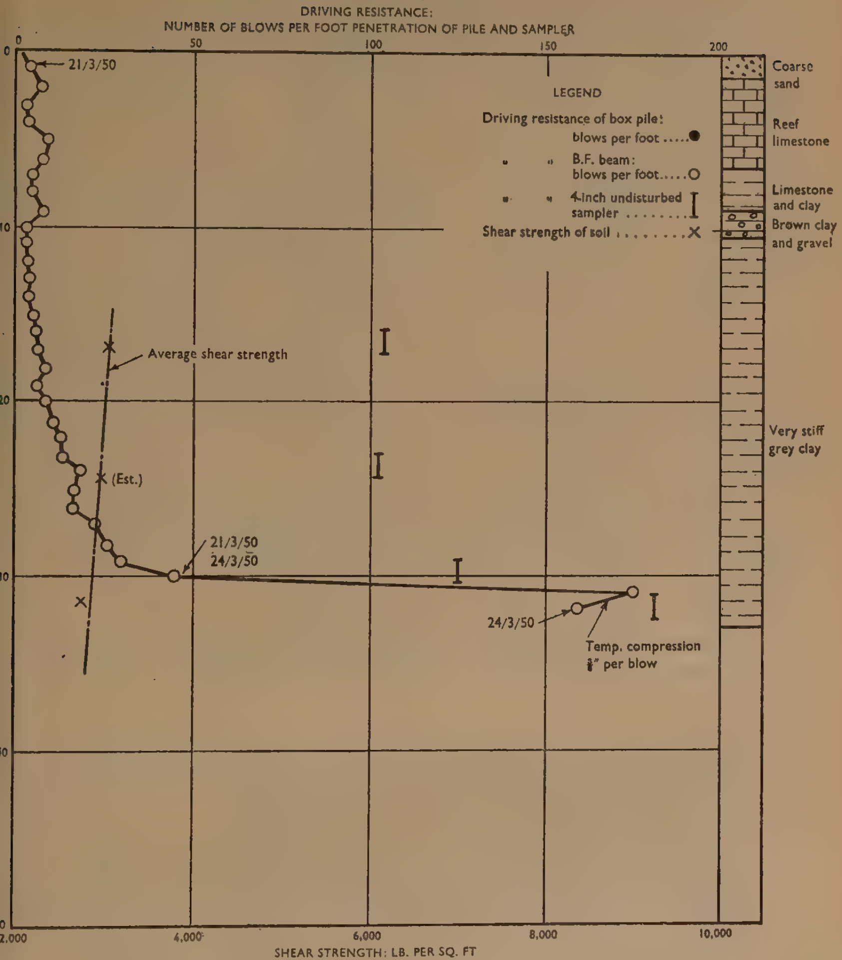
TEST PILES NOS 3 AND 4

Pile No. 3.—Larssen B.P.3 box pile, open end: Length: 65 feet. Weight: 2·5 tons. Total penetration: 25 feet 10½ inches.

Pile No. 4.—Broad-flanged beam, 12 inches by 12 inches. Length: 70 feet. Weight: 2·58 tons.

Hammer.—3-ton drop-hammer falling 3 feet. Timber dolly.

Fig. 15 (*c*)



Test Pile No. 5

Pile No. 5.—Broad-flanged beam, 12 inches by 12 inches. Length: 70 feet. Weight: 2·58 tons. Total penetration: 31 feet 8 inches.
Hammer.—3-ton drop-hammer falling 3 feet. Timber dolly.

In order to correlate the test-pile data with the borings, the piles were driven close to borehole positions where differing soil conditions were found, and the comparative driving resistance of two types of steel section pile was investigated.

The test piles were made up from 30- and 35-foot lengths of Larssen

B.P.3 box-piles welded together on site to form four 65-foot-long box-piles, and 35-foot lengths of 12-inch-by-12-inch-by-82·55-lb. broad-flanged beams, fish-plated and welded on site to form two 70-foot-long piles. Driving resistance and temporary compression were measured from a platform supported by lengths of 4-inch-diameter borehole casing driven into the sea bed.

Driving records are compared in *Figs 15* (*a*), (*b*), and (*c*) with the soil strata and laboratory test data from boreholes close to each pile position.

The driving resistance of the 4-inch-diameter British Standard sampling tube driven by a sliding hammer on the end of the square-section drill rods is also shown, indicating a rough correlation with the pile-driving resistance. The interruptions in driving were caused partly by winter squalls and gales, and partly by deliberate attempts to investigate the likelihood of " take-up " in driving resistance. The effects of " take-up " can be compared in various types of soil after intervals of several days between stages of driving. The broad-flanged-beam piles showed a lower driving resistance than the box-piles driven at the same locality (that is, piles 3 and 4). Both types of pile have the same cross-sectional area of steel, and both can be regarded as butt-ended piles, owing to the plug of soil formed at their lower ends and carried down with the piles. However, the gross cross-sectional area of the box-pile is the larger, thus giving a higher end resistance.

The driving resistance through the upper limestone layers was surprisingly low, confirming the penetrating power of the steel section pile in hard strata, as experienced in similar conditions at Mina-al-Ahmadi.

Ore Loading Pier, Sierra Leone

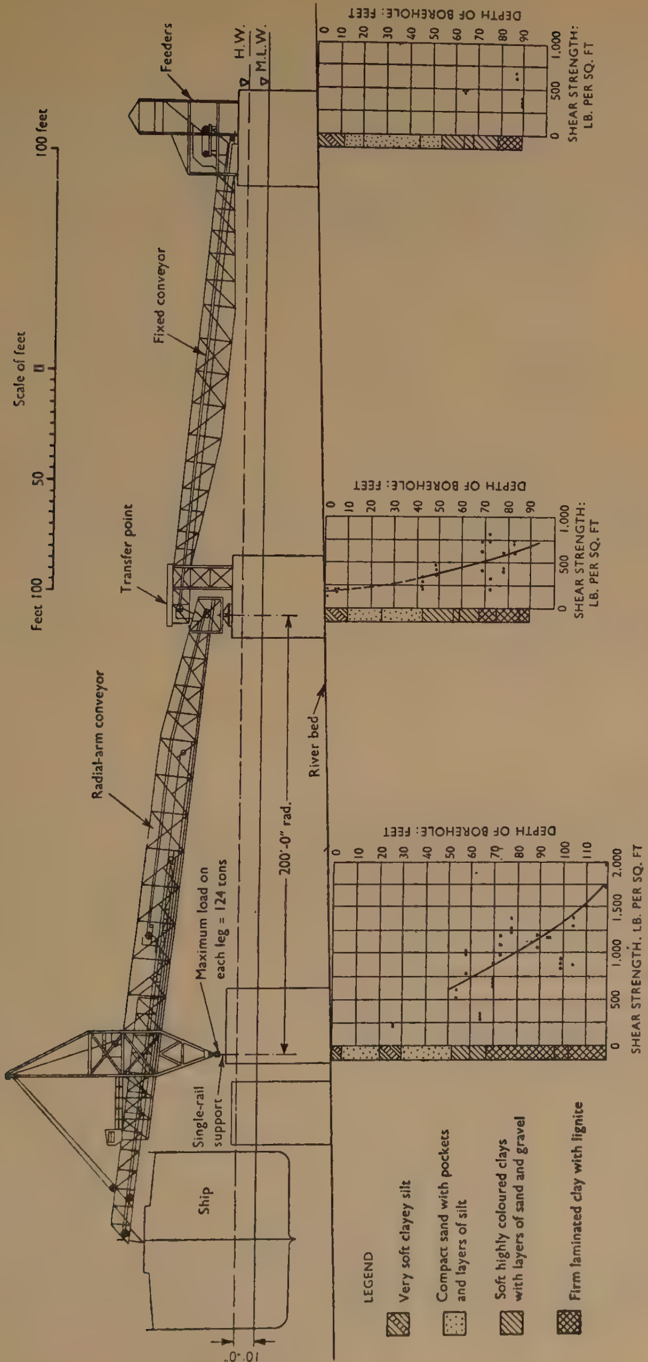
Borings were made for the Sierra Leone Development Company in 1952, in the tidal estuary of the Rokel River at Pepel, for an ore loading pier. The boring rig was mounted on a coaling lighter from which boreholes were sunk to 150 feet below the river-bed. Difficulty was experienced in pitching and driving the first few lengths of casing in the 4-knot current at ebb tide, and this stage of the work was generally done at slack water.

A section through the borings is shown in *Fig. 16*. For the purpose of analysing the foundation design problem, the deep river deposits were grouped into :

(*a*) An upper layer of soft silts and sands with some well-defined stratification.
(*b*) An intermediate stratum of soft multi-coloured clays.
(*c*) A lower stratum of firmer clays containing layers of sands, lignite, and transported laterite.

The variation in shear strength with depth of the clay layers is shown in *Fig. 16*. Most of the shear strength tests were made in a site laboratory. For the foundation analysis, 30-foot- and 15-foot-diameter caissons, and

Fig. 16

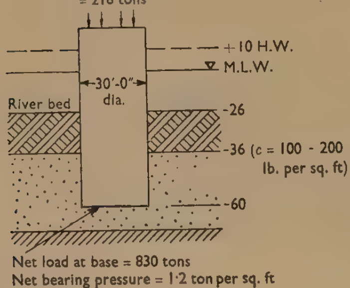


Longitudinal Section

groups of piles, were considered. The foundations of the transfer point from the fixed conveyor to the radial arm are shown in *Figs 17* (*a*) and (*b*) for the alternatives of a caisson or a pile group. The bearing capacity of a layer of soft clay beneath the bearing stratum of compact sand was a critical factor in the analysis. Although both types of foundation were calculated to have an adequate safety factor against shear failure in the sand or underlying soft—firm clay, the piled foundation was preferred, since its lighter loading would reduce consolidation settlements.

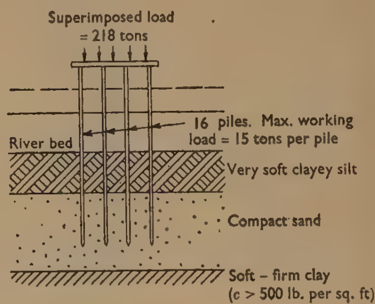
Figs 17

(*a*)



Cylindrical Foundation

(*b*)



Piled Foundation

As part of the site investigation, a study was made of the condition of the jetties and other structures in the locality. Greenheart piles had been reduced from 12 inches to 6 inches in diameter by marine borers, and steel structures were heavily coated with molluscs and barnacles which damaged bituminous coatings. Because of this, steel mooring buoys had to be scraped and recoated every 12 months. Most of the outer steel plates of a 20-year-old cylindrical pier were corroded away between high and low water, and a steel spike could be worked for a foot into the softened and disintegrated concrete behind the plates. A reinforced-concrete piled jetty, belonging to another concern in the area, partially collapsed as a result of attack between high and low water after only 10 years of service.

Analysis of river-water samples showed a marked difference in the total dissolved solids at high and low water (see Table 1), indicating that the river contained about 50 per cent of sea-water at high tide. From analyses of the river-water and disintegrated concrete, it was thought that deterioration was caused by a combination of attack by sulphates in the sea-water and by acids derived from floating organic matter. Corrosion of steel structures was aggravated by the warmth of the water (80–84° F.) in the dry season, and the presence of floating organic matter. These factors would also accelerate attack by borers on timber structures.

It was concluded that high-quality dense concrete was the most suitable constructional material for the new pier.

TABLE 1.—CHEMICAL ANALYSES OF SAMPLES OF RIVER-WATER

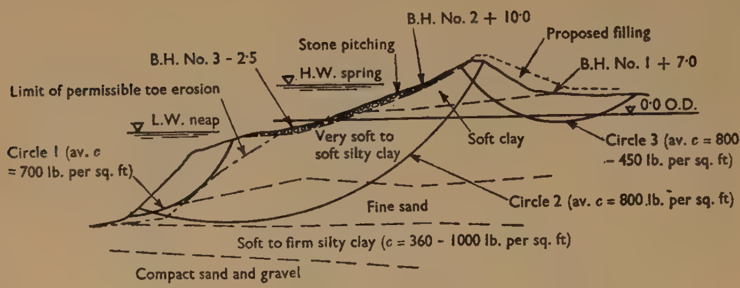
Sample	No. 1. High tide, mid-stream: parts per 100,000	No. 2. Low tide, mid-stream: parts per 100,000
Suspended solids	3·68	1·56
Fe_2O_3 and clay	3·25	1·12
Organic matter	0·43	0·44
Dissolved solids	1,536·90	239·15
CaO	24·00	5·40
MgO	75·97	12·20
Na	489·01	74·03
Cl	856·00	127·00
SO_3	64·69	14·51
HCO_3	8·30	4·64
pH (as received)	7·1	7·2
pH (after boiling)	7·5	7·3
Possible composition of dissolved solids:		
$Ca(HCO_3)_2$	11·03	6·17
$CaSO_4$	48·96	7·93
$MgSO_4$	54·00	14·81
$MgCl_2$	136·87	17·10
NaCl	1,242·56	188·29

Sea Wall at Coryton, Essex

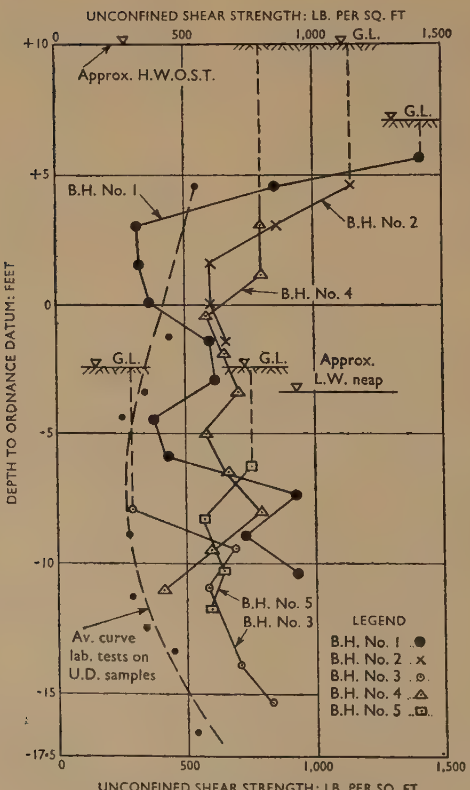
The stability of the existing sea wall at Coryton Refinery was investigated for the Essex Rivers Board in connexion with a proposal to widen and raise the levels of the bank. Borings were made on the bank by shell and auger methods using tubular steel scaffolding stagings for the work below low-tide mark. Vane shear tests were made in the soft clay layers adjacent to the boreholes. A cross-section of the river bank, the vane-test results, and the critical slip circles are shown in *Fig. 18*. It was evident that some erosion of the toe had taken place and the estimated safety factor of the lower slopes was 1·3 (circle 1). Large quantities of slag filling had been dumped on the lower slopes to check the toe erosion on this part of the sea wall.

After raising the height of the bank the safety factor of the overall slope was estimated to be 1·7 (circle 2). This would be reduced to 1·4 (circle 2) if further erosion of the toe took place to the limits shown. After raising the bank, the safety factor of the rearward slope was estimated as 4 (circle 3). In the major flood of January–February 1953, the bank was breached owing, it was thought, to erosion of the rearward slope after overtopping.

Figs 18



ANALYSIS OF SLIP CIRCLES



VANE AND LABORATORY SHEAR TESTS FOR BOREHOLES ON SEA WALL

SEA WALL AT CORYTON

Investigation of Cliff Erosion at Cullercoats Bay, Northumberland

Severe erosion and undermining of the cliffs at Cullercoats Bay had taken place, causing falls and encroachment on house property at the top of the cliffs. An investigation was made for the County Borough of Tynemouth in 1951, in connexion with a proposal to extend the existing wall which was undermined, and to support the exposed cliff face. Eight borings were made at the foot of the cliffs in the position shown in *Fig. 19*, using a light air-driven diamond core-drill, the air compressor being stationed on top of the cliffs. Seventeen distinct strata of the Upper Coal Measures, consisting of sandstones, mudstones, shales, coals, and fireclays, were passed through in the borings, as shown on the longitudinal section in *Fig. 19*. The most extensive erosion had occurred at the places where the friable and laminated shales and coal seams out-cropped at beach level in the vicinity of boreholes 3 and 6. This had resulted in undermining and collapse of the overlying and more resistant sandstones. The extent of the undermining at borehole 4 can be seen in *Fig. 20* (facing p. 229). Erosion may have been accelerated by " patch " workings of the coal seams. A 2-foot-6-inch deep cavity was found in the coal seam in borehole 7, and a shaft existed close by. It was concluded that the new sea wall should be founded at depths sufficient to prevent erosion by sea action of the friable shale, coal, and fireclay strata, and that special attention should be given to the foundation conditions in areas where abandoned coal workings were suspected to be present.

Acknowledgements

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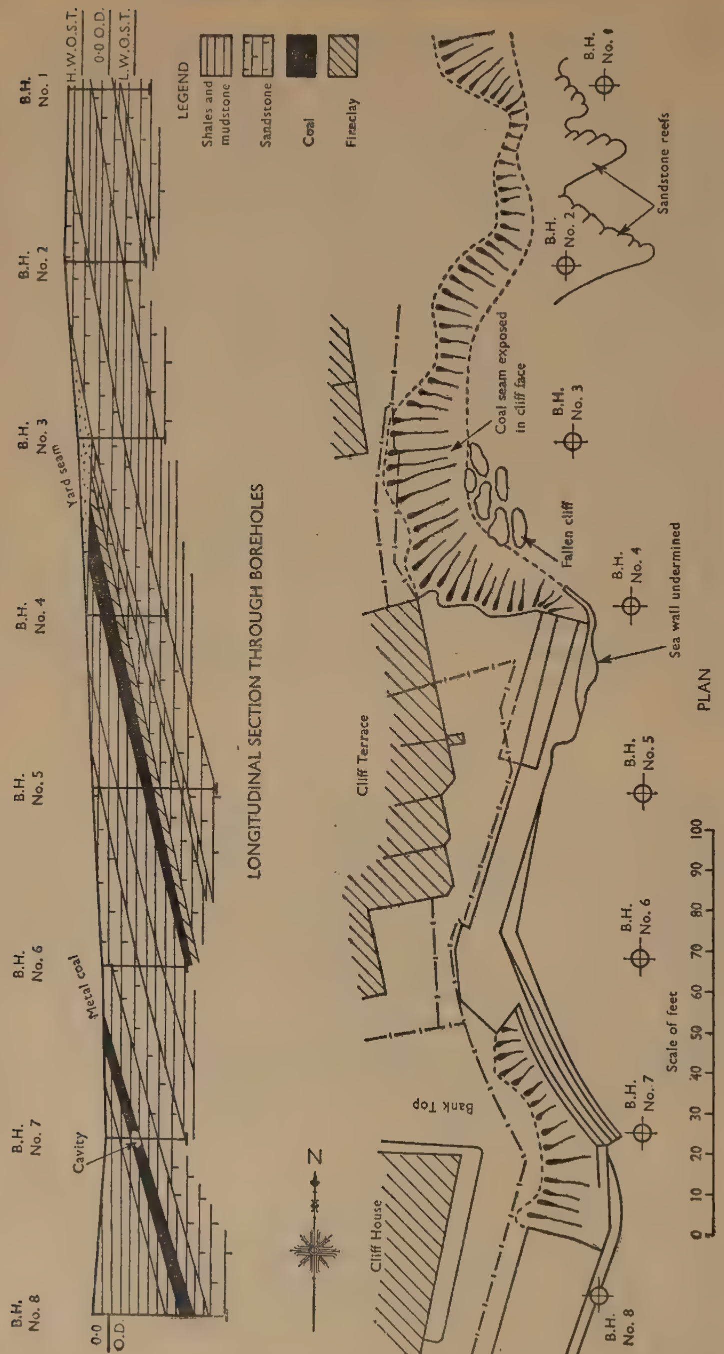
The site investigations described in the Paper were carried out by the Central Laboratory of George Wimpey and Company Ltd, under the general direction of Dr L. J. Murdock, M.Sc.(Eng.)., A.M.I.C.E.

Figs 19



CLIFF EROSION AT CULLERCOATS BAY, NORTHUMBERLAND

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The Paper is accompanied by seven photographs and nine sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

Mr P. R. Robinson observed that his firm had been associated with two of the sites mentioned in the Paper, and he could substantiate the statements which the Author had made on the subject generally. It was hazardous work and cost a good deal more than borings on land.

One aspect of the subject which Mr Robinson thought deserved emphasis was that the underwater survey and the subsoil investigation were closely related. The underwater survey had to be as accurate as possible,

and in his opinion the best way to do it was by echo sounding, which was much more accurate than ordinary soundings. It gave an actual picture of the sea-bed and would disclose features which otherwise might be missed. He remembered a case in point in the Gulf of Cutch, where an echo-sounder survey of a creek bed showed that the bottom was of saw-tooth formation; that might not have been discovered by ordinary soundings. It should be borne in mind, however, that echo-sounder surveys were usually carried out for navigational purposes, and so the results as plotted tended to show rather less water than actually existed. That, of course, was all right for shipmasters, but might be a little misleading, to the extent of perhaps a foot or so, to civil engineers.

He thought that the fact had to be accepted that borings from floating craft could be carried out only in comparatively calm weather. That was true even when using spudded craft, the main advantage of which was that work could be continued in less favourable weather conditions. However, spudded craft might be dangerous in waves of a certain amplitude, and the passage from shore to craft and back might be hazardous in rough weather. Furthermore, if the craft was not large enough to ride out a storm, either shelter must be available or provided nearby or the craft must be withdrawn to port when bad weather developed—thereby losing much time.

Mr Robinson agreed with the Author that wash borings might very well be used more often, and thought that the Code of Practice was too severe in that respect. He also agreed that often where a large area had to be surveyed, a grid of core borings was satisfactory, with wash borings interspersed between. The latter, carefully interpreted, could be very valuable indeed.

Site investigations of the kind in question had to be undertaken much in advance of the main contract work and might require a great deal of administrative organization, and whilst contractors were well accustomed to look after themselves in such matters, consulting engineers were sometimes not altogether alive to the fact that they might have to do their own housework under such conditions.

Mr K. G. Benham confined his remarks to drilling problems. In connexion with the 2-cwt anchor mentioned on p. 228, the Author had no doubt been referring to average conditions where the weather might be relatively mild, but he had mentioned Immingham. Mr Benham had also had experience with boring from craft in the Humber, and he did not think that 2-cwt anchors would have served very well there. As mentioned in the Paper, a length of chain on each anchor was essential. Its length should be at least six times the tidal lift, so that the bulk of the chain never left the bottom; it was really serving as an additional anchor. It was nearly always necessary to have six anchors.

A cantilevered platform was usually essential, but some of the most satisfactory borings of which he had had experience had been done from a

heavy craft with a well in the middle. The Port of London Authority had one such craft, but it was not always available. At the other extreme he would put a craft which he had recently seen operating; it was a schooner type with the platform in the bows, which struck him as a very unsatisfactory arrangement.

Fixing casings to the craft, in order to get the benefit of the tide to lift it again, had been found to be a somewhat risky operation, depending on the type of craft. With a vessel of 150 or 200 tons displacement it was probably satisfactory.

Mr Benham could not agree with the Author's interpretation of the Code of Practice as being that percussion boring methods—as distinct from shell and auger—were "unsuitable for site investigational boring." Mr Benham did not think that that was quite what the Code of Practice meant; it said that they were not so suitable as shell and auger boring "for *very careful* site investigation work." For instance, in the case of the borings in the Thames, for the Dartford-Purfleet Tunnel in 1927, the examination of what lay above the chalk became of secondary importance to the actual level and nature of the chalk itself; the latter was not damaged in any way by the use of chisels and there would have been no other way of penetrating it.

Mr A. C. Scott showed a series of lantern slides of a few types of craft which had been used for work of the kind described in the Paper. He said that the cost of bringing to the site the most suitable craft very often precluded their use, and the engineer had then to exercise considerable ingenuity and a high degree of improvisation in adapting local craft for his purposes.

By means of a slide Mr Scott illustrated an Arab schooner of about 360 tons, with the rig mounted nearly amidships. The work was carried out offshore and required quite a heavy vessel. The anchors had been mounted as usual in all four quarters, and use had also been made of the vessel's main anchor. Diesel power gave the boat an advantage over the traditional barge, which had to be towed from point to point. The Arab crew lived on board, and it had proved to be a very suitable vessel for the purpose.

Fig. 4, facing p. 228, illustrated a case in which an attempt was made by the contractor and the consulting engineer to do forward planning. The quarter anchors and the very neat deck lay-out can be seen in the figure. Owing to the sloping sides of the "Beetles," which were a couple of Mulberry Harbour units [16] it had not been possible to make full use of the deck space, but they had used a prefabricated decking which could be dropped into position between the two vessels, and it had been an efficient and quick method of getting to work. The pontoons themselves had been shipped to Aden as deck cargo and had been used for the new oil-harbour works at Little Aden.

[16] See reference 4, p. 255.

When using the traditional dumb barge, the rig was cantilevered over the side. There were two schools of thought on this matter, some people holding that it was better to put it in the bows or stern, and others favouring putting it over the side. Mr Scott's firm had felt that cantilevering over the side gave better all-round working space. The roll of the vessel was felt, but the amplitude of the roll was not so great as if the rig had been mounted in the stern or the bows where the rise and fall was much greater. Such a vessel had been used off Kuwait for the new harbour works built there some years ago.

On a site at Little Aden, Bailey type bridging units had been available, and had been used to improvize a twin floating raft of very wide beam. It had been ideal for the work, but he would give a warning against using it in all cases. At Little Aden there was a very small rise and fall of tide—only a few feet. It did not give an opportunity of having a slot through which one could pull away from the casing at night, or allow it to ride out a storm. A hole in the decking had to be cut and was completely trapped by the stringers and main girders of the raft itself. However, it was an excellent vessel in certain limited conditions, such as obtained in rivers where some tidal movement and current could be tolerated. Mr Scott said that a point against its wide use in Britain was that it was necessary to transport to the site the six pontoon units, which, if R.E. trucks were not available, meant using at least three lorries and another lorry was needed for the decking, and with the present high cost of road transport that was an inducement to use local craft, even if they had certain drawbacks.

For boring work off Shoreham, in connexion with new harbour works there, an 180-ton powered vessel had been used. The boring platform had been cantilevered out at the side. The reason for using such a large powered craft might be a little obscure. In fact it had been required by the harbour authority that if any vessel navigating Shoreham harbour got into difficulties they would have to pull out. The harbour master did not like the fact that they had to leave the casing behind, and that had proved a bone of contention throughout, but it could not be helped.

Anchors were usually laid by manhandling from a small fishing vessel about 15 feet in length. The tendency was to use a rather lighter anchor than would otherwise be employed because of the handling. It was necessary to have a vessel large enough to handle a 5-cwt anchor and a long length of chair. Three men had to lift it over the side, working in considerable danger and difficulty, so that there was a tendency to use a lighter anchor on this work than would otherwise be the case.

Mr Scott urged all engineers who required borings to be made to try to site boreholes in the middle of existing stagings. One often found a red blob on a plan an irritating distance beyond the end of an existing jetty, and to make a boring in that position demanded a very heavy cantilever,

which was very costly. Several more borings could be provided at the same cost if they were specified within the existing work.

Mr Jack Robertshaw thought that the Author had underrated the usefulness of geophysical methods. In Mr Robertshaw's experience, geophysical surveys over water had proved to be just as successful and valuable as geophysical surveys on land. By means of a slide Mr Robertshaw illustrated a resistivity survey over a dam site in Fiji. On the site in question four electrodes had been suspended in the water from four ropes which stretched from one bank to the other ; the electrodes had then been moved across the ropes, and in that way the resistivity had been measured at various sections across the river. The water was a layer which could be allowed for in the interpretation. The current was very fast and the river subject to frequent heavy floodings, which made river borings very hazardous, but the investigation of the site had been successfully carried out by inclined boreholes from each bank and by the resistivity survey across the river.

In the case of another resistivity survey for a dam site in Africa, the river was far wider and it had not been possible to suspend ropes across it ; the electrodes had, therefore, been suspended in the water from small native canoes, and the instruments operated from a raft.

Unfortunately, the resistivity method was not suitable for marine investigations, because seawater acted as a short circuit and prevented penetration of the current into the ground. However, the seismic refraction method was suitable for both river and marine surveys, and was generally more accurate than the resistivity method. In the seismic method a shot was fired close to the surface, and the vibrations penetrated through the overburden and were refracted along the top of the rock surface. Those vibrations re-emerged through the overburden and were picked up by vibration detectors spaced along the ground surface. The signals from the vibration detectors were fed back to an amplifier unit, and the amplified signals were recorded by a seismic camera. In that way a seismic record was obtained, from which it was possible to measure the time taken for the vibration waves to travel from the shot point to each detector. By plotting those measurements as a graph the depth of the refracting rock surface could be calculated. The basic equipment required for a seismic survey was very portable and consisted of vibration detectors, an amplifier unit, a shot-firing set, and a recording camera.

In a seismic survey for a dam site in Pakistan 2-lb. charges of gelignite were exploded in the river and vibration detectors placed on each bank. Several shots were fired across the sections of the river, and in that way it had been possible to determine the depth of the rock along each section. In all, forty sections had been investigated on the site, covering the proposed sites for the main dam and the cofferdams. The survey results were confirmed by subsequent river borings.

During a marine seismic survey for a proposed oil jetty in Scotland a

small charge of gelignite had been fired at each end of a section on the bed of the sea with ten vibration detectors placed on the sea-bed between the two shot points. From the seismic records it had been possible to determine the rock profile beneath the sea-bed along the section. Borings had been made at the offshore end of each section investigated, and a considerable saving in cost was achieved by having a seismic survey in conjunction with the borings.

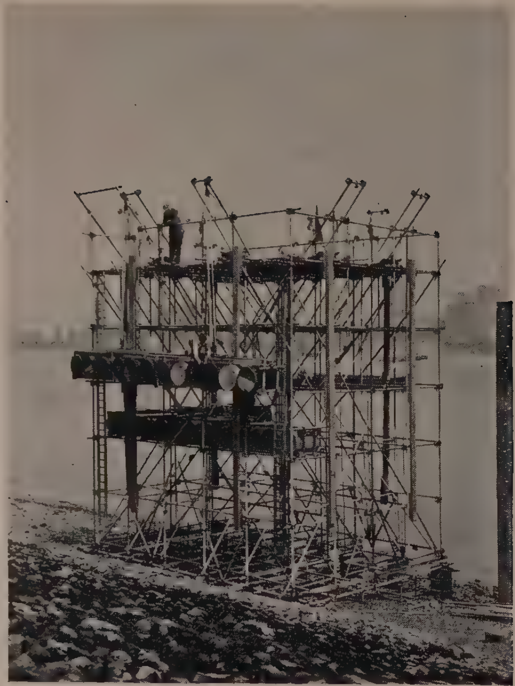
In conclusion, Mr Robertshaw said that he would like to refer to the sonic or echo-sounding method to which reference was made in the Paper. That method was normally used for sea-bed contouring, but it had been claimed that it could be of use for determining the depth to bedrock beneath the sea-bed. He understood, however, from the manufacturers of the equipment in the United Kingdom that they made no such claim for their equipment. It had on occasions picked up rock at a very shallow depth when the rock was overlain by soft mud.

Mr I. K. Nixon remarked that, whilst boring on land normally followed an orthodox pattern, boring over water often introduced unusual problems both in the planning of the work and during its execution. He showed a series of lantern slides to illustrate his point. Any work which was carried out from a floating craft was expensive, especially in tidal waters, and site investigation work was no exception and so staging was always to be preferred if possible.

A particular problem arose when borings were required between high and low water. It was occasionally more economical to use floating craft rather than to construct a series of individual stagings, each of which had usually to be provided with a separate approach way in order to bring the equipment over the position of the boring. That, however, was not always so, because there were times when the river- or sea-bed was covered with large rocks or wreckage, which endangered the craft when it grounded. In other cases the bed might be steeply shelving, or it might be of soft mud, which prevented access to the craft when grounded at low water. In those circumstances, a staging which could be made to float might provide the most economical solution to the problem. An example was illustrated in *Fig. 21*. It was often constructed on the site between high and low water, so that it could be easily moved into position. Some buoyancy was normally built into the staging by using 40-gallon oil drums or some other form of tank which could be easily sealed. The additional buoyancy necessary to make the staging float was either added to the side of the staging at low water or brought into operation in some other manner.

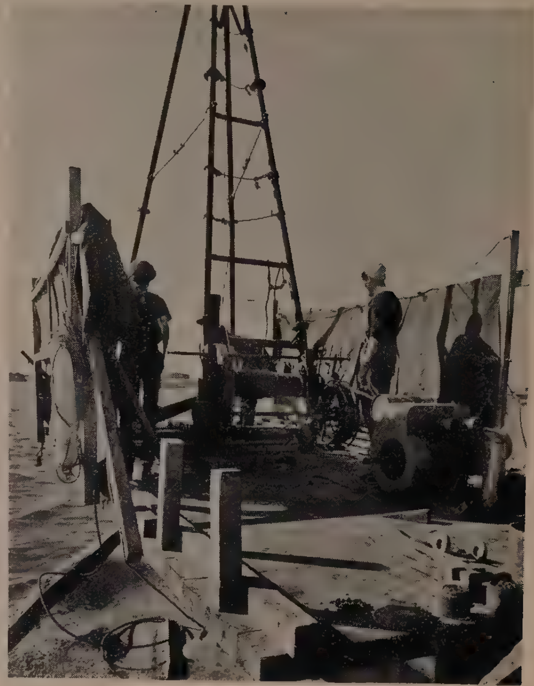
When boring over water in a busy river, it was usual to employ a dumb barge as the floating craft. Usually the rig was mounted at one end, as shown in *Fig. 22*, for that provided a more stable platform, and thus assisted, as the Author had mentioned, in extracting the tubes should it become difficult to withdraw them. He believed that that was probably a more important consideration in deciding the position where the

Fig. 21



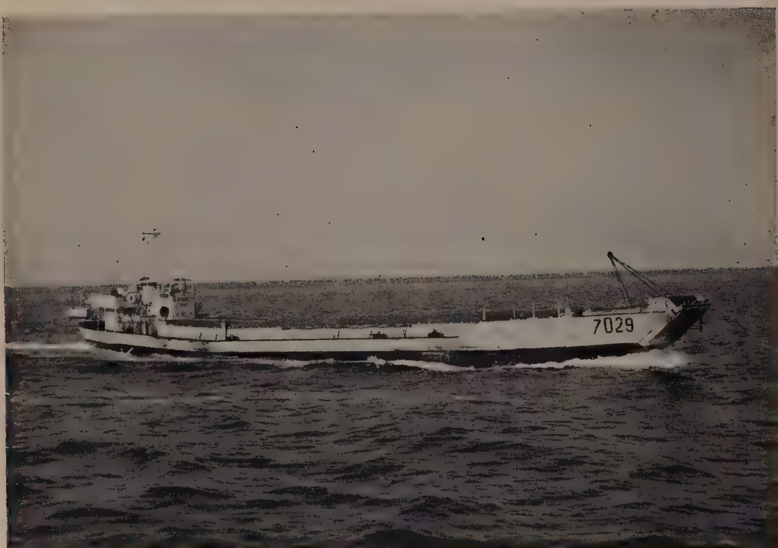
Typical Example of Floating Staging

Fig. 22



Bow-Mounted Boring Rig in Dumb Barge

Fig. 23



LANDING CRAFT WITH RIG MOUNTED ON LANDING-STAGE FLAP, AS USED IN SYRIA IN 1948

rig should be placed. The hold of the barge was usually loaded with ballast in order to make the craft more steady in a choppy sea and also to cut down the effective area which could be acted on by the wind and so cause movement during the boring operations.

Investigations offshore, as distinct from those in rivers or close inshore at sea, were probably the only occasions when self-propelled craft were advantageous. Moving and positioning a dumb barge was slow, and the consequences of a storm could be serious. On the other hand, a self-propelled craft could be moored in the position of a boring and stay in position right up to the last moment before taking up its own anchors and withdrawing to a sheltered position.

The Author had mentioned the use of landing craft, and *Fig. 23* showed one of two vessels which had been used off the coast of Syria in 1948. So far as Mr Nixon was aware, it was the first occasion when such craft had been used for that form of work. In each craft the rig was mounted permanently in position on the landing-stage flap as shown in *Fig. 23*. By that means a very convenient platform was available for working; it could be raised quickly and easily whenever the sea became too rough for working or during moving.

Mr F. L. Cassel said that it was important to establish the sea- or river-bottom level at the site of the borings, and that was often difficult. Where possible, the best method was to erect a gauge board somewhere near the shore which could be correlated with the level of the casing at the boring site.

An important feature of underwater survey work was the consideration which frequently had to be given to shipping, which might reduce the actual working time very considerably. It was therefore frequently economical to work two shifts, or even round the clock.

Mr Cassel mentioned some of the in-situ tests to which the Author had referred. Except in a case where there was a very considerable amount of exploration work to be done, and where qualified personnel had to be present in any case, in-situ tests were usually very much more expensive than laboratory tests, mainly because delicate machinery was required. The foreman of a drilling crew could be trained to take samples or carry out standard penetration tests; but, for vane tests and static penetration tests, it would as a rule be necessary to have qualified personnel—laboratory assistants or engineers—at the site, perhaps only to carry out two or three tests a day, and that was a very expensive matter. Nor was it certain beyond all doubt that in-situ tests always supplied better or more reliable results than laboratory tests.

Mr Cassel had carried out standard penetration tests since 1949, and he had sometimes obtained, in fine or silty sands, blow figures as high as 300 for 9-inch penetrations. Terzaghi and Peck, however, had described every soil involving more than 50 blows as very dense. In his latest book, Peck mentioned up to 70 blows, but nobody had mentioned more than

70 blows for 12-inch penetration. Mr Cassel was aware that in very fine silty sands the penetration figures were not perhaps directly related to their densities, because of the pore pressure set up in those sands owing to their low permeability.

Similarly, he had tried to obtain penetration figures when taking undisturbed 4-inch cores in the usual way. He had found no proper correlation, and he wondered whether it was possible to obtain proper correlation between the penetration figures for 4-inch barrels and the 2-inch spoon of the standard penetration tool.

With regard to the vane test, he had observed that in very soft clays the clay stuck to the rod, in spite of copious greasing, and that made the calculation of the shear strength doubtful. Had the Author had that experience ?

The vane test had recently been used successfully in the United States of America for a very large-scale operation from a barge across a shallow lake ; the object had been to determine the shear strength of varved clay to a depth of 90 feet below the bottom. The method was distinguished by the fact that no casing had been used. The vane was moved upwards and downwards between tests by jets, using water under pressure.

Mr R. E. Tuckwell referred to experience which he had recently gained in the Persian Gulf area, working offshore, where they had been investigating a jetty design. Both drilling and pile tests had been involved and a pontoon had been equipped so as to serve both purposes. The pontoon had been made of tank sections similar to the Mulberry Harbour "Beetles," but with the added advantage that they could be made up to any shape and any length, and that they had flat top decks.

Six 5-cwt anchors had been used ; that was near the limit which could be raised and repositioned with the 100-horse-power launch. Most dumb barges required moving frequently from place to place, and therefore the raising of the anchors and their repositioning was important, and the smaller the anchor the more easily could the operation be done. Elsewhere a 2-cwt plough-type anchor had been used on a barge which was approximately 70 feet long by 20 feet beam, and it had ridden out the worst storms in Scottish lochs without giving any anxiety.

The pontoon which Mr Tuckwell described had been constructed from self-contained airtight tanks, bolted to one another and strengthened with steel angles and joists. Between the tanks there had been a 9-inch gap which had proved useful when boring. The pontoons could be easily assembled on shore by completely unskilled labour, and floated off at high tide. The addition of a pile frame and five winches had converted the pontoon into a floating pile-driver. A smaller pontoon, of similar construction, had been used for ferrying out the test piles.

The 80-foot steel piles were pitched in 60 to 70 feet of water by slinging from one end whilst levering the free end off the barge. That method

could be used only for steel piles, since single-point lifting would damage timber or concrete piles.

During fine weather, core borings had been taken through the 9-inch gap between tanks in the centre of the pontoon.

Mr Tuckwell said that when carrying out the pile testing, some offshore borings had been made from fixed stagings. The driving of the piles which formed the staging had also yielded information for pile-driving data. The work had been done about $\frac{1}{4}$ mile offshore, and the staging had withstood winds of force 8 on the Beaufort scale (approximately 50 miles per hour).

Mr Tuckwell referred to the necessity of ascertaining the uplift of a pile. A test pile driven in the centre of a dolphin had been withdrawn by means of a hydraulic jack operated by a pump on the platform of the staging. In that way it had been possible to measure the load and movement accurately. In the case of barges with a displacement of 150 tons or more, their buoyancy could be used to withdraw piles or casing tubes from the sea-bed, but that method could not give an accurate figure of the actual load required.

The Author had mentioned that steel piles were much more flexible for construction.

By means of a slide, Mr Tuckwell illustrated the damage which was sustained by the toes of piles in driving. The test piles shown, which had been withdrawn using an extractor, had been given only sixty blows for a penetration of approximately 10 feet, but in that time they had sustained heavy damage. True knowledge of the condition of the foot of a pile was not often obtained when piles were being driven, because most piles were left in or cut off at the sea-bed after a test.

Mr Tuckwell thought that Arab divers could be useful in investigations in Middle East waters since they could descend to depths of 60 feet without air equipment.

Mr C. W. N. McGowan referred to the survey carried out in the early spring of 1948 at the site of the then proposed oil pier at Mina al-Ahmadi.[17] The site of that pier was a very open one; the pierhead stood about 4,200 feet from the shore, and the nearest sheltered water was 30 miles away. Sudden squalls could make the surface of the sea really rough in half an hour or so.

Preliminary test piles had revealed the existence of a rocky sea-bed on the line of the pier, and it had been essential during the early part of the design period, to ascertain the soil conditions, with special emphasis on the thickness and extent of the rock layer. Accordingly, the consultants had received instructions at the end of December 1947 to engage a specialist contractor with appropriate boring equipment and personnel to carry out the investigation.

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[17] C. W. N. McGowan, R. C. Harvey, and J. W. Lowdon, "Oil Loading, and Cargo Handling Facilities at Mina al-Ahmadi, Persian Gulf." Proc. Instn Civ. Engrs, Part II, vol. 1, p. 249 (June 1952).

During the survey, five deep holes and twenty-seven shallow wash-borings had been made, in addition to one deep hole and ten wash-borings within the limits of a proposed harbour. Samples of the strata penetrated in the case of two of the boreholes had been obtained, but poor weather conditions prohibited the further use of lining tubes for procuring samples, owing to the severe time-limit which had been imposed for the carrying out of the survey. Therefore the variations in the characteristics of the strata penetrated had been considered on the results of penetration resistance to a jet pipe and from the specimens collected by the tools.

It had been found that, although the upper and lower surfaces of the rocky sea-bed were hard, the core contained soft pockets. It proved to be a formation of recent origin, and had been colourfully described by the Geological Museum as " Sand grains, and fossils—mainly gastropods and brachiopods or lamellibranchs—embedded in micro-crystalline calcite with possibly a small pyrite content." At one borehole a weighted chisel had been sufficient to effect penetration, whilst at three others charges of up to 6 lb. of gelignite had been necessary. Beneath the rocky layer there was a comparatively soft material, comprising sand interposed with chalk beds. Penetration had been effected by means of a water jet, using a pipe of $1\frac{1}{2}$ inch outside diameter and pressures generally of 75 lb. per square inch, with occasional rises to 150 lb. per square inch.

The survey had given rise to doubts as to whether the 14-inch-by-$14\frac{1}{2}$-inch-by-73-lb. steel H-piles could be driven through the hard layer by the normal use of the piling hammer. In the event, the heads and toes had to be strengthened by welding additional short steel pieces at each end (see *Figs 44* in reference 17).

Mr McGowan then read some extracts from the notes which the Author had prepared while in charge of the work. These extracts emphasized a point which Mr Robinson had made, namely that working under sea conditions could be extremely hazardous. The first extract read :—

> " We got the tool revolving on the rock bed but we couldn't see any downward movement of the tool, so we sent the diver down after about half an hour's revolving, and he reported that the tool hadn't made a mark on the rock surface (the sand is less than an inch thick and the motion of the tool had washed it all away into the strong current). The rock surface was level but rough, and the shot dropping down the tool is immediately washed away from beneath the cutting edge by the current. The result is that the shot cannot be brought into use until the tool has penetrated at least four inches, and the shot has a chance to stay down at the cutting edge."

The second extract read :—

> " On the 21st February we moved into position for the third sea borehole and put down the jet pipes which revealed a few inches of

sand over hard rock. The diver went down and reported 9 inches of sand over the rock. He put down a 3 lb. charge which hardly marked the surface, tried another which stabbed it for an inch or two, and yet another which made a pocket about 4 inches deep. So I decided to use a rotary table, and two hours steady drilling got us down a little over 2 feet in hard rock, using the 2-inch tungsten cutter. We were in about 55 feet of water at high tide and this meant using 65 feet of rods. There were indications from the behaviour of the tool in its last few minutes of drilling and from material brought up inside the core barrel that we were through the hard rock and into sand or soft rock."

The third and last quotation read :—

"We have had three days of really hard work, but not much boring progress to report, since two of these days have been spent in salvaging the jetting boat which was sunk during a heavy gale last Thursday night. Winds were still very strong and seas running as high as I have seen out here, and so we were not able to commence salvaging operations until the next day, when we brought the barge over to the jetting boat moorings and anchored her stern over the sunken craft (in 20 feet of water). The operation took all day, with much manoeuvring with ropes to clear the rudder of the barge and reslinging to get a more direct lift when she was awash.

"After this day's effort you can imagine our dismay the next morning when we got aboard the barge and found the jetting boat sunk once more, alongside and half underneath the barge, where we had tied her up the night before. It was once more a job of slinging and hauling up from winches, after which we towed her to the jetty and all our men were working in relays throughout last night keeping her baled dry."

*** **Mr Rudolph Glossop** observed that the Author seemed a little sceptical of the value of geophysical methods in civil engineering investigation. That attitude had been very general a few years ago, but in view of recent improvements in technique it was no longer justifiable.

Mr Glossop's firm had, in the past 3 years, carried out nine seismic refraction surveys on civil engineering sites, and five of them had later been checked by boreholes. The results showed that that method could be expected to give the depth to bedrock within 5 or 6 per cent of its true value, which was sufficiently accurate for any purpose. It should be remembered that the great advantage of the geophysical method was that it gave a continuous bedrock profile, so that, unlike investigation by boring, there was no chance of a buried channel being missed. The firm had recently carried out thirty-eight surveys by the resistivity method, and

*** This and the following contribution were submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

of them sixteen had been required for the investigation of dam foundations. Of those sixteen, five had been checked by borings and showed a percentage error rather higher than in the case of seismic work, the mean value being about 8 per cent. Nevertheless, the results were satisfactory from a practical point of view.

The Author had mentioned the use of the echo-sounding method for site investigation. So far as Mr Glossop knew, echo sounding had never been used with success for that purpose, nor was it likely to be. In echo sounding the time was measured for a small pulse to travel from a transmitter on the survey craft to the sea-bed, and to be reflected back to a detector device. The depth could then be found, since the velocity of sound in water was known. The energy of that pulse was far too small to enable it to penetrate the bed of the sea to any depth, and any device likely to succeed would have to produce, in each pulse, energy equal to the explosion of $\frac{1}{2}$ lb. of gelignite, as was used on seismic refraction surveys for shallow depths. Even assuming that such a device was possible, there would be a further difficulty where bedrock was overlain by unconsolidated sediments as was generally the case, for it would then be necessary to know the speed of transmission in the sedimentary material, which might vary between 3,000 and 8,000 feet per second. To measure those velocities it would be necessary to have recourse to a seismic survey after all.

In the discussion one speaker attributed the penetration-vane device to the Americans. That was incorrect. The original vane testing apparatus sprang from a suggestion made by Dr L. F. Cooling, and had then developed simultaneously in Sweden and in England. The penetration vane apparatus had been invented by Mr Glossop's colleague, Mr Offer, and had been regularly used by his firm for 3 years.

Mr K. J. Birkett observed that contractors were inclined to launch craft for sea-bed investigations with additional high superstructures, such as pile frames, boring rigs, and fixed out-rigged staging, without full knowledge of the maritime code controlling stability, classification, or the operational range for dumb or self-propelled vessels in open or exposed waters. Such craft, unless fitted out according to the Ministry of Transport, or appointed agents, can result in dangerous marine conditions, increase the hazards of boring, and subject the crews to unnecessary operational and marine dangers.

Few contractors realized that certain types of craft, whether they were stationed in England or overseas, were at all times under the jurisdiction of the Ministry of Transport and could not be altered or fitted with specialized plant without sanction from the local authority appointed by the ministry.

The Ministry of Transport's cover was as important to the contractor as a marine insurance, and so long as all their recommendations were carried out, the chance of a major catastrophe was reduced to the minimum.

The Author had stated that anchors should be fairly light and weigh about 2 cwt each, and had suggested that individual moorings required a separate hand- or power-driven winch or the use of a multi-drum unit. That was a very controversial question, and until it could be proved beyond any doubt that vertical power-driven capstans and warping heads using a much heavier form of anchor would hold a craft in a true and accurate position, then the method described by the Author, and used extensively throughout the civil engineering profession for river and sea work would continue to be used. It was natural that the normal contractor would not purchase expensive capstan equipment that had a limited use, whereas the hand- or power-driven winch could be usefully employed on other forms of constructional work. The weight, size, and type of anchor depended on the size and shape of vessel, the holding ground, and expected weather within the working area, and not on the fact that it could be handled by a number of strong men, perched on the bow of a service launch.

Recently, during a large sea expedition along the south coast of England, two types of craft had been used, and it was interesting to compare the speeds of moving and positioning. The first type of craft was the normal conventional dumb craft rigged with five heavy mooring winches and 2,000-lb. stockless anchors, carrying drag chain and 60 fathoms of soft-heart flexible steel wire mooring rope. The complete unit had been serviced and attended by a 250-horse-power Diesel-driven tug. The other unit was a self-contained tank landing craft equipped with one main central electric 70-ton capstan, and two auxiliary warping heads. That unit carried only three 1-ton anchors, and two light C.Q.R. ploughs, which were used only if a cross-current was inclined to alter the ship's working position. The difference between the outputs of the two units was that the tank landing craft could complete between 18 to 20 cycles of work in 10 hours, against the 5 to 9 cycles completed by the dumb unit in 10 to 12 gang-hours.

A second comparison was afforded by the craft used on the north coast of Syria and described by Mr Nixon. Each of those craft had moved, on an average, twice per day, during which time they had carried out two 70-foot-6-inch wash borings, including the taking of disturbed and undisturbed samples. Each craft had carried only one vertical capstan plus three 15-cwt stock- and stockless-type anchors, which had been dropped and picked up by the same capstan unit.

The Author had mentioned that accurate positioning was often the most difficult operation when setting the craft to shore markers. That might be the case in rough weather, or if the boring position was some distance from the shore. From Mr Birkett's experience the most satisfactory method for quick and accurate positioning for offshore work was to compute the true sextant bearing incorporating the markers; then to drop a small concrete sinker with a coloured pellet buoy, attached to a

light cable, slightly longer than the sounding within the boring position. The pellet buoy served a dual purpose, for it indicated the positions and directions in which moorings, ground tackle, or anchors should be laid.

The Author's company had carried out a similar investigation along the south coast of Syria during the same period, with nearly identical ground conditions, but using a 350-ton Arab schooner. Could he give any comparative rates of progress, against that of the landing craft performance ? It might be stated that mobile craft were expensive to equip, run, and maintain for the smaller investigation ; that might be, but it was felt that an early result obtained from the field was most important, for it allowed the planning and design of the proposed project to be advanced, and the site constructional difficulties recognized at an early date.

Although the landing craft was not the right type of craft to use on all sites and was limited in use, its running costs were comparable with those of the normal dumb unit and tug ; it had the added advantage of being able to operate 24 hours a day, and house and feed operational crews with a certain amount of comfort, as against the discontentment and danger so prevalent among boring crews working on exposed dumb units.

Had the Author used the large-type outboard motors known as the "Harbourmaster" which enabled a dumb craft to be moved and steered into position without the aid of a tug or launch, or the sectional flat bottomed self-propelled "Z" craft ?

The Author had mentioned that spudded craft were essential for rotary blast-hole drilling, and could be used for borings. For marine boring that type of craft would be ideal, so long as all boring was done on a fairly hard and level sea-bed instead of some on a graduated bank, which was generally the case for jetty sites. Spudded craft for blast holes would undoubtedly be satisfactory for the first firings, but after that, the craft would be sitting partly on pinnacles and shattered rock debris that would gradually settle under the weight of the foot-bearing plate, thus throwing the craft and rotary drill rods out of alignment. Mr Birkett thought that any form of rotary drilling, of sufficient size to take a suitable charge, would be very slow and expensive compared to cable drilling, which could be carried out from a suitable floating craft, even in a choppy sea.

During the early part of 1949, very successful experiments using cable tools had been carried out near the mouth of the river Tees, over an area of hard marl, which it had been considered could be removed by blasting. A series of 6-inch-diameter holes had been sunk to a depth of 20 feet below sea-bed level, charged, and fired. Unfortunately, the scheme had had to be abandoned for it was feared that the blast would disturb a tunnel constructed within the vicinity.

The Author had described wash-boring procedure, consisting of small-diameter tubes, applying a comparatively low pressure, with a small

quantity of water. Using the small-size tubes made it impossible to recognize the small and large gravel sizes, for they were either broken up before they could enter the annular space between the two tubes, or pushed to the outside of the casing. By using larger sets of tubes with much higher volumes than those recommended by the Author, much harder formations could be penetrated and greater depths obtained. Furthermore, undisturbed samples of the cohesive soils of standard diameter could be extracted for the necessary laboratory tests.

The Author, in reply, agreed with Mr Robinson that the underwater survey and subsoil investigation were closely related, and that they could, with advantage, be undertaken simultaneously. The echo-soundings were not only more accurate, but could be made at much greater speed than ordinary soundings, and if, as mentioned in the Paper, the echo-sounding method could be developed to give information on the stratification of the subsoil, that would be a very great advance in the technique of marine site investigations.

Although spudded craft of the braced staging type could be designed to be stable in rough weather conditions, it could be dangerous, and was often impossible, to board the staging from the service craft in such conditions. For boring in the open sea, the Author's firm always tried to find a craft sufficiently seaworthy to ride out bad weather. That was particularly desirable in squally conditions, since it could be a long job to raise all the anchors and make for the nearest harbour, particularly when working with native crews who were often slow in understanding and movement.

Mr Benham had referred to a craft with a centre well. The Author had used such a craft in the Thames Estuary, and it was satisfactory for work in sheltered water where there was no strong current. The risk with a centre well was that, with a fast falling tide and strong current, the casing could rise up in the well and the craft move off position, causing the casing to bend over or break off, with possible damage to the bottom of the vessel. The Author always preferred to be able to warp the vessel quickly away from the borehole casing at times of sudden changes of current or wind direction. It was preferable to have the boring platform amidships to set out the boring gear. The deck space at the bows and stern was (as shown in *Fig. 22*) often very limited and encumbered with capstans, winches, and other gear. When utilizing the buoyancy of the vessel to raise the casing, there was no need to make the lift from the boring platform. The vessel could be manœuvred to bring the casing under the bows or stern to make the lift. The Author agreed that there were conditions in which percussion boring methods (as defined in the Code of Practice) could be used.

Mr Scott's contribution amplified the information given in the Paper on the use and handling of floating craft. The precise siting of holes in relation to existing works, such as jetties, could have an important bearing

on the cost of site investigations, and a boring made a few feet inside an existing piled jetty could give just as reliable information as a borehole made from a platform cantilevered out from the face of the jetty. However, care should be taken to avoid siting boreholes close to existing piles, in view of possible disturbance of natural subsoil conditions caused by the penetration of the piles.

Mr Robertshaw and Mr Glossop had said that the Author under-rated, or was sceptical of, the usefulness of geophysical methods. That was not quite the case, but the Author felt that such methods had a very limited application in site investigations for marine and river work. Geophysical methods gave only the level of the interface between several well-defined strata, but did not give any indication of the engineering characteristics of the various strata. Maritime civil engineering works nearly always involved problems concerning stability of structures, such as piled jetties, dock walls, embankments, and the like. For an economical solution of those problems, the engineer needed information on the shear strength of cohesive soils, the particle size and density of cohesionless soils, and the occurrence of ground-water. Sufficient data on those characteristics could not be obtained from the widely-spaced boreholes used to correlate geophysical surveys, and increasing the number of boreholes would tend to make a geophysical survey unnecessary. If the investigation was made merely for the purpose of finding out the depth of soft material over rock or boulder clay, such as in a dredging investigation, the method of putting down a large number of jetted probings could be as quick and economical as geophysical surveying. The Author noted that a number of the geophysical surveys mentioned by Mr Robertshaw and Mr Glossop had been made on the sites of dams and, although check borings had been made, final judgement on the usefulness of the method should wait until the excavation of the cut-off trenches was completed.

Mr Robertshaw and Mr Glossop had doubts on the feasibility of the echo-sounder to explore subsoil conditions below the sea-bed. The accounts of the method as used in the United States of America (references 12 and 13, p. 255) seemed to give an encouraging view of its possibilities, and although the Author had also been told by the British manufacturers of the equipment that it was not claimed that the apparatus would explore conditions below the sea-bed, some research could usefully be done in that direction.

Floating staging of the type illustrated by Mr Nixon could often be useful as an intermediate stage between fixed stagings extended from the shore and floating craft. Floating staging was particularly useful in shallow tidal water where a barge might suffer damage on grounding. The Author did not agree with Mr Nixon's assertion that the problem of extracting the casing determined the position of the boring platform. The casing could be lashed on to any part of the vessel when it was being pulled up. The necessity for self-propelled craft could be over-emphasized.

A launch was needed, in any case, for servicing the work and laying out anchors. The launch could act as a tug to a dumb barge, and in rough weather take off the crew, leaving the barge to ride out the storm.

Mr Cassel mentioned the time spent by qualified engineers in making in-situ soil tests. It was often advantageous to make all the in-situ tests towards the end of the job when information was available on the general soil conditions, and the most favourable sites for the in-situ tests could be selected on the basis of that information. The work could be done with greater expedition by adopting tests which did not require to be made in a borehole, for instance the cone penetration test or vane test, using the penetration-vane equipment. Mr Glossop claimed that the latter equipment, referred to by Mr Cassel, had been invented by a colleague. In a newly-developed science, such as Soil Mechanics, where there was constant interchange of up-to-date information, the Author deprecated claims to "inventions" of apparatus, since parallel and equally good advances were often made by a number of investigators at the same time, and many investigators made and used such apparatus without considering the improvements to be of sufficiently far-reaching importance to warrant publicity. The difficulty of clay sticking to the vane rod mentioned by Mr Cassel could be overcome by calibrating the apparatus, using a rod turning in the soil without a vane at the end of it. The Author had not experienced exceptionally high results with the standard penetration test, except occasionally, where a large piece of gravel had been taken down by the mouth of the sampler, or where a cemented sand-layer had been met. No correlation had been made between the number of blows required for penetration of the 4-inch sample tube and the 2-inch split-spoon sampler. However, if the penetration figures given in *Figs 15* (*a*) and (*b*) were compared with the shear strength of the clay, and the comparison was extended to published data giving the range of shear strength for various standard penetration numbers, it would be seen that the penetration resistances of the 4-inch tube driven by a 275 lb. hammer and the 2-inch sampler driven by the 140 lb. hammer were roughly the same for a given shear strength. That might be coincidental.

Mr Tuckwell's slides had shown the expensive equipment needed to drive and load test piles on off-shore sites. He had been fortunate in being able to withdraw his test piles.

Mr McGowan had brought back very vividly the Author's experiences of the difficult work at Mina-al-Ahmadi. That job had compelled the use of a craft which could ride out a storm, since, as mentioned by Mr McGowan, the nearest shelter was 30 miles away.

Mr Birkett had given a word of warning about high superstructures mounted on floating craft. Stagings cantilevered from the side of a vessel with a low freeboard could also be dangerous if the vessel rolled sufficiently to cause the staging to dip into the sea. Selection of sizes and types of anchors was a compromise between a number of factors, not the least of

which was the limiting weight that could be handled by the service launch. The Author's firm had used a landing craft on the Syria site in addition to the Arab schooners, and there was no doubt that progress with the former was faster, owing to its greater stability and powered capstan, but it was also more expensive to hire and maintain. The Author had no experience of large outboard motors or " Z " craft for site investigational work, but considered such craft could be useful in some circumstances. The essential feature of spudded craft was the ability to adjust the length of individual spuds to cope with varying sea-bed levels and tidal conditions. Therefore, the Author could not see that the difficulties mentioned by Mr Birkett could arise. The Author had seen numerous references in technical publications to rotary blast hole drilling for underwater rock excavation, using multiple drill boats, but had seen no references to cable rigs used for such work on a large scale.

The Author would not advocate wash boring in gravelly soils. With large-sized tubes, there was a tendency for large pieces of gravel to accumulate at the bottom of the hole, requiring high pressures to wash them to the surface, thus re-introducing the evils of wash boring, as mentioned in the Code of Practice.

The closing date for Correspondence on the foregoing Paper was the 15th May, 1954. No contributions received later than that date will be printed in the Proceedings.—Sec. I.C.E.

RAILWAY ENGINEERING DIVISION MEETING

23 February, 1954

Mr W. K. Wallace, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Railway Paper No. 52

"Railway Civil Engineering in the United States of America"

by

Ian Macdonald Campbell, B.Sc.(Eng.), and Norman John Nicholls, B.Sc.(Eng.), A.MM.I.C.E.

SYNOPSIS

The Authors have recently spent a year in the United States, studying civil engineering on American railways, and this Paper is based on their experiences.

The organization of the Association of American Railroads and the American Railway Engineering Association is described, followed by a discussion of the organization of the railway civil engineering departments.

Track structure is described in detail, including rail sections, rail joints and fastenings, sleepers, ballast, formation, and earthworks. Particular mention is made of the systematic testing of rail, long welded rail, rail-end hardening, manganese crossings, and methods used in stabilizing track.

The most outstanding feature of American railways is the almost complete mechanization of track maintenance. The machines used for ballast cleaning, tamping, and the "production line" method of rail renewal are fully described.

The more common forms of bridge construction, the organization of bridge and structure maintenance, and the methods and equipment used by the A.A.R. for bridge testing, are noted.

Brief mention is made of various other railway installations, including freight classification yards, motive power depots, goods sheds, and stations.

The Paper concludes with a comparison of the conditions affecting civil engineering on British and American railways, and with suggestions for future developments in Great Britain.

Introduction

This Paper is based on the experiences of the Authors during their visit to the United States of America under the Technical Assistance Programme of the Mutual Security Agency, in co-operation with the Ministry of Education. The visit lasted for a year from September 1951, the first 4 months being spent at the University of Illinois in the study of subjects allied to railway civil engineering, and the last 8 months with a number of railway companies and the Association of American Railroads Central

Research Laboratory, in the study of maintenance and construction practices.

It has been the endeavour of the Authors to present a general picture of practices in the United States but, although Recommended Standards are published by the American Railway Engineering Association, the railway companies do not adhere rigidly to them. Throughout the Paper, British technical terms and the nearest equivalent British designation for personnel have been used; for example, Roadmasters and Track Supervisors have been described as Permanent Way Inspectors.

The Organization of the Association of American Railroads (A.A.R.) and the American Railway Engineering Association (A.R.E.A.)

There are more than a thousand separate railway companies in North America, and the A.A.R. is an association of the major companies, its funds being derived from proportional contributions from the member companies. Its activities cover all aspects of railway operation including finance, law, transportation, engineering, public relations, and many others.

The A.R.E.A., on the other hand, is an association of individual railway civil engineers and other interested persons. The A.R.E.A. publishes a manual of Recommended Practices and Standards, prepared and periodically amended as necessary by a number of committees composed of A.R.E.A. members. There are twenty-three of these committees covering the various aspects of engineering work; their recommendations are considered and ratified by the members of the Association at the Annual Convention.

Much of the research behind the recommendations of the committees is carried out by the A.A.R. Central Research Laboratory in conjunction with universities and the research departments of the railway companies. The research projects are recommended by the various A.R.E.A. committees and sanctioned financially by the A.A.R.

The A.A.R. Central Research Laboratory consists of a number of departments including the engineering, mechanical, refrigerator-car, container, and sanitation divisions. The work of the engineering division is wide and varied and includes rail testing, rail stresses, rail joints, the seasoning and treating of timber sleepers, the mechanical wear of sleepers, chemical weed killers, road-bed stabilization, bridge testing, etc. The results of the work of the engineering division are published in A.A.R. Bulletins and then in the Proceedings of the A.R.E.A.

Organization of the Railway Civil Engineering Departments

Most companies employ an organization in the civil engineering department similar to that in use in Great Britain. The position of the Chief

Engineer varies from company to company, but normally he reports to the Vice-President (Operations). The Chief Engineer's responsibilities include, apart from civil engineering work, the maintenance of electrical and mechanical equipment other than that contained in motive power and rolling stock. The Signal and Telegraph Engineer is responsible to the Chief Engineer.

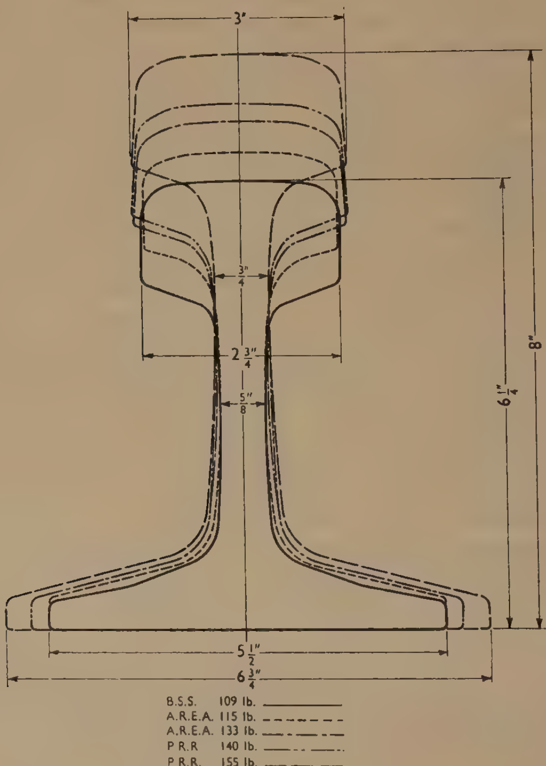
The Chief Engineer has, under him, Assistants in charge of bridges, track, new works, electrical engineering, etc., together with the necessary design staff. The amount of design work carried out in a head office is generally less than that carried out in an equivalent office in Great Britain because, in the United States, there are fewer structures per mile of track and they are generally of more recent construction; there is normally sufficient space to lay-in standard track arrangements without resorting to special designs, and most of the detailing of special trackwork is left to the contractor who fabricates the material. Day-to-day maintenance of track and structures and minor new works are carried out by the District Engineers with a staff of Permanent Way and Works Inspectors. Major new construction schemes may be carried out either by direct labour or by contract and are supervised by a Resident Engineer appointed for the job. The main difference between this type of organization and that in Great Britain is in the dual responsibility of the District Engineer, first to the Chief Engineer for technical matters, and secondly to the Superintendent for financial and policy matters; the latter is responsible for all operational, commercial, and technical matters in his area.

One of the largest companies, the Pennsylvania Railroad Company, employs a different organization. The Company's territory, which extends from the East Coast to Chicago and St Louis, is divided into three Regions. The Chief Engineer of the system is responsible for technical policy and overall supervision of the department. On each Region there are two Chief Engineers: one is called the "Chief Engineer," and carries out all new design and construction within the Region and is responsible to the Chief Engineer of the system; the other is called the "Chief Engineer, Maintenance of Way," and is in charge of the maintenance and renewal of track and structures and is responsible mainly to the Regional General Manager and to the Chief Engineer of the system for purely technical matters.

The organization of districts is similar to that described above, with the exception that the majority of Permanent Way Inspectors are qualified engineers. University graduates entering the Company's service as Permanent Way Apprentices under the training scheme, which normally lasts 3 years, receive general experience of railway engineering with the main emphasis on permanent way. At present, owing to the shortage of qualified staff, they are usually appointed as Assistant Permanent Way Inspectors after about 18 months. In this position they advance from branch lines to main lines before being appointed as Permanent Way

Inspectors, and thence to Assistant District Engineer and so on through the Engineering Department; or, at any stage, if found suitable, they may be transferred to the Operating Department from which they can advance to the highest managerial positions. This policy is in line with that being followed successfully in many American industries, namely, of appointing technical men to management. This scheme has the merit of attracting

Fig. 1



Comparison of Rail Sections

ambitious technical men to the Company's service, but it also has the disadvantage that, in emphasizing the maintenance of permanent way to the almost total exclusion of its design and the design and construction of bridges and structures, it produces senior engineers lacking experience in these important subjects. Graduates for design and construction are engaged through similar training schemes but have rather less chance of promotion to the highest ranks.

Fig. 2

Rail Test Car

Fig. 3

Rail End Hardening—Electric Induction Heating

Fig. 4

TYPICAL JOINT WITH 36-INCH FISHPLATE

Fig. 9

SWITCH POINT GUARD RAIL

Fig. 11

1-in-20 Manganese Insert Crossing

Fig. 12

Solid Manganese Crossing

Fig. 13

Self-Guarded Frog

Fig. 14

The Hursh-Reigh Ballast Tamper (P.R.R.)

Track Structure

On American railways, emphasis is placed on the maintenance of the permanent way, since this constitutes the greatest proportion of the assets. In the following description of the track structure and its maintenance, the generally adopted practice is given without considering all the variations.

(1) *Rail.*—The flat-bottom rail is in universal use, ranging in weight on main lines from 100 lb. to 155 lb. per yard (*Fig. 1*), the latter being used by a few companies only. The normal weight for heavily used main lines is 132 lb. The standard rail length is 39 feet, although there is a trend towards the use of a 78-foot rail if rolling difficulties can be overcome. The A.R.E.A. specification for the chemical composition of rail steel includes 0·69 to 0·82 per cent carbon and 0·70 to 1·00 per cent manganese, depending upon the weight of rail. All rails are graded and colour-marked at the ends according to quality.

The introduction of controlled cooling for rail steel has virtually eliminated rail failures caused by shatter cracks. The main sources of rail failures at present arise from detail fractures resulting from a shelly rail condition, and from fatigue failures in the joint area. The number of rail failures occurring in service has been considerably reduced by the introduction of systematic rail testing by track-mounted test cars (*Fig. 2*). There are a number of different types of these test cars in use, employing either electro-magnetic or electro-inductive principles. Rail flaws are picked up by a search coil and automatically recorded. This test may be used as frequently as once a month on main lines, and faulty rails are immediately removed from the track. The record of the incidence of flaws over a length of track is given consideration when a re-laying programme is being planned. These test cars are unable to detect faults in the joint area although the electro-magnetic car can get closer to the joint than an electro-inductive car. Inspection of the rail ends for flaws is done by a portable type of supersonic detector. This instrument has proved to be very efficient and can be operated by one man and a helper.

The practice of welding rails into continuous lengths of a mile or more has increased recently, with apparent success. Most commonly used is the gas pressure-weld, in which method the rail ends are brought together at a pressure of 2,700 lb. per square inch and heated uniformly by gas jets. Pressure is maintained until a shortening of $\frac{7}{8}$ inch is obtained ; joints are then normalized by heating to about 1,500° C. Rail head surfaces are ground down and the finished joint is tested by the magna-flux method. A very successful flash-weld method is being given consideration at the present time.

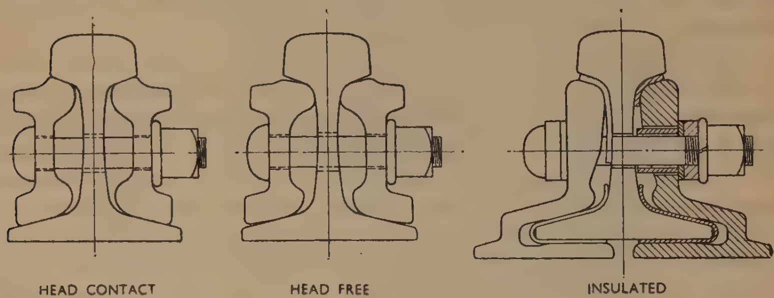
Much corrosion is caused to rails by the brine drippings from refrigerator cars, which form a high proportion of the freight moved. It has become the practice on some railways to spray the web and bottom

flange of the rails with oil about once a year. Although this does not improve the appearance of the track it retards corrosion.

(2) *Rail joints.*—Rail joints are staggered by half a rail length which, it is asserted, cuts down noise and gives a smoother ride by reducing the "bump" associated with square joints. In addition, the opinion is expressed that square joints on a curve would make maintenance of alignment difficult. The Authors rarely experienced severe rolling on passenger trains and it was noticeable that, on a number of goods lines with low joints, springing of goods vehicles was adequate to allow a safe speed of 50 to 60 miles per hour.

Rail-end batter is a serious problem and is frequently the main consideration in the renewal of the rail. It has been found economical to employ welding teams to build up joints to a regular programme. More

Figs 5



TYPICAL FISHPLATES

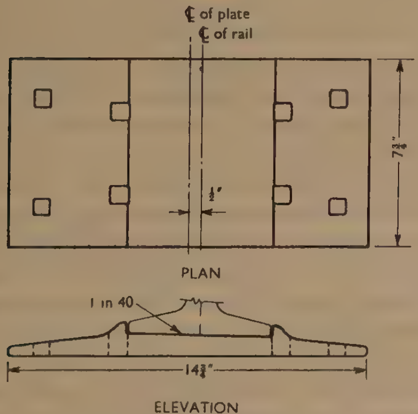
recently the practice of rail-end hardening has gained popularity and is now almost universally adopted. The running surface of the rail ends may be hardened either at the time the rail is laid in the track by gas or electric induction heating and air cooling (*Fig. 3*, facing p. 276), or they may be hardened in the rolling mills by gas heating and air quenching. The former method allows adjacent rail ends to be ground to the same level before hardening, which is considered by some to be of paramount importance. The latter method allows a more accurate control of heating and quenching and some steel firms are increasing their plant with the ultimate aim of hardening all rail ends. The Brinell hardness of the rail ends after treatment is about 350.

Two types of fishplates are in general use for main lines: the four-hole 24-inch to 28-inch plate, and the six-hole 36-inch or 38-inch plate (*Fig. 4*, between pp. 276 and 277). Some companies consider the latter to be an excellent plate for heavily used main lines, whilst other companies consider that the 24-inch plate is adequate for all lines. The top surface of the fishplate may either bear against the underside of the rail head (head contact) or against the fillet between head and web (head free) (*Figs 5*).

Fishing surfaces are heavily greased before new plates are fixed and some companies do little or no subsequent lubrication. A number of different types of joint packings have been tried and opinions differ as to their merits. One type of grease cake was widely used until it was found that rail failures at the joint were increasing, this being attributed to water which got trapped on the rail web and upper fillet, and induced corrosion fatigue. Improvements have been made and this type is regaining popularity with some companies. Other methods of joint lubrication consist of brushing on to the outside of the fishplates an oil which works behind them, or of spraying oil into the fishing surfaces with a power spray. Both these methods are considered to be sufficiently effective. With the use of these methods of lubrication the need for the annual removal of fishplates is avoided.

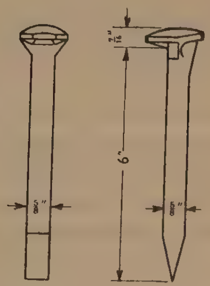
(3) *Baseplates and fastenings.*—Baseplates are of rolled steel with a seat canted at 1 in 40 to meet a worn wheel tread with a 1 in 20 wheel

Figs 6



Standard Baseplate for Straight Track

Figs 7



Standard Cut Spike

coning; all plates now used have twin shoulders. Earlier types of baseplates were too small and cut into the sleepers; the present-day plate for 132-lb. rail is approximately 14¾ inches by 7¾ inches or 8¾ inches, punched with six or eight square holes for the fastenings (*Figs 6*). It has been found that the distribution of pressure under a baseplate having no eccentricity on straight track is not uniform, being greater outside the rails than on the running-edge side. This results in differential cutting of the plate into the sleeper, which, if not rectified, may result in wide gauge. The rail seat is therefore set eccentrically on the plate by ¼ to ⅜ inch. The difference of pressure becomes very much more marked on curved track where it is recommended that an eccentricity of 1¼ inch be used. Details

of A.R.E.A. standard plates for curves, giving this eccentricity, have been published, but the plates are not yet in general use.

The standard fastening most widely used is the steel cut spike, with a $\frac{5}{8}$- or $\frac{3}{4}$-inch-square shank, tapered at the end and with a slightly rounded head (*Figs 7*). The length under the head ranges from $5\frac{1}{2}$ to $6\frac{1}{2}$ inches. These spikes are used in two positions: first, as line spikes driven down on to the rail base to hold the rail in position; and secondly, as "hold-down" spikes to fasten the baseplate to the sleeper. It will be apparent that this type of spike can exert little or no positive hold down on the rail, since the first wave motion of the rail after driving the spike will withdraw it a small amount. The only effect of the line spike, therefore, is to maintain the rails to gauge. This is in accordance with the principles of American track construction, in that free vertical movement on the baseplate seat is allowed, whilst the baseplate is fastened rigidly to the sleeper. Thus the principal wear due to the vertical motion of the rail takes place between rail and baseplate, whilst cutting of the plate into the sleeper is minimized and movement of the sleeper in the ballast, with the consequent liability to pumping, is reduced.

Various experimental installations have been made using spring fastenings for holding down baseplates, but none of these has so far received approval. To reduce plate cutting into the sleeper and to keep out moisture which hastens cutting, various types of rubber and fibrous pads have been used with varying degrees of success.

The use of rail anchors is regarded as essential and the standard minimum recommendation is to fix eight forward anchors and two back-up anchors per rail length on lines carrying traffic in one direction only, with eight forward and eight back-up anchors on lines with traffic in both directions. Continuous welded track is heavily anchored, particularly towards the ends of the welded length, where anchors may be placed on both sides of every other sleeper.

(4) *Sleepers.*—Timber sleepers are used exclusively, experiments with reinforced-concrete sleepers having been unsuccessful. The standard sizes of sleeper for main lines are 8 inches by 7 inches by 8 feet 6 inches, or 9 inches by 7 inches by 8 feet 6 inches, sleepers being placed 24 to a 39-foot rail-length. The 8-foot-6-inch sleeper gives a 38-per-cent greater bearing area of sleeper per yard of track than is obtained in Great Britain, and therefore a more nearly continuous support to the rail. The main timbers in use are oak, gum, Douglas fir, and various species of pine. Sleepers may be seasoned naturally in air, by boiling in creosote mixture, or by a new patent vapour-drying process. All sleepers are treated, generally with a mixture of creosote and petroleum oil or creosote and coal tar. Petroleum and coal tar are comparatively cheap, increase water-repelling properties, and retard evaporation of the preservative. A retention of 8 to 10 lb. per cubic foot of the mixture is desirable for a soft wood.

It is now considered that the mechanical wear of sleepers, not deteriora-

tion, is one of the major determining factors in their renewal. Efforts to reduce mechanical wear by pads have been mentioned above, and it is common practice to improve sleepers which are heavily checked or split before treatment, by the insertion of bolts, dowels, or S-irons at both ends. The practice of spacing sleepers relative to rail joints varies widely; some companies space sleepers to ensure a suspended joint, some to ensure a supported joint, whilst others place sleepers at approximately 20-inch centres regardless of joint position.

(5) *Ballast, formation, and earthworks.*—All types and gradings of ballast are encountered, including crushed rock (the most common for heavy main lines), lava ash, pit-run gravel, processed gravel, and blast-furnace slag. Although gradings up to 3 inches maximum have been used in the past, the present tendency is to reduce this to 2 inches or even 1½ inch. Depth of ballast below sleeper level varies considerably, because most of the track may be raised periodically on the old ballast without interference from overhead structures, which are few and far between.

Track drains of the conventional channel or pipe type are comparatively rare except in built-up areas. In open country, the width of the railway right-of-way is frequently sufficient to allow a wide ditch to be cut by a bulldozer or other mechanical equipment, this ditch being re-dug periodically as silting occurs.

Formation trouble resulting from soft spots, pumping, etc., is frequently encountered. A common remedy used for this is formation grouting. This process has been used on a large scale, with apparent success, both for track formation and for slope failures, the initial cost of the grouting being paid for by measurable reductions in maintenance during a period of 3 years. The grout strength may range from 1 : 1 to 12 : 1 or even weaker, it being thought that the amount of cement has little effect on the efficiency of the grout, apart from ensuring a free flow of sand through the grout pipes. In order to assist the flow, the sand should not be too sharp, and fly-ash or a bituminous lubricant may be added to the mix. The grout is injected, either by a mud-jack for large continuous operations or by pneumatic pressure for smaller intermittent jobs, through pipes driven to a depth of approximately 5 feet below sleeper level for normal track formation work (*Figs 8*).

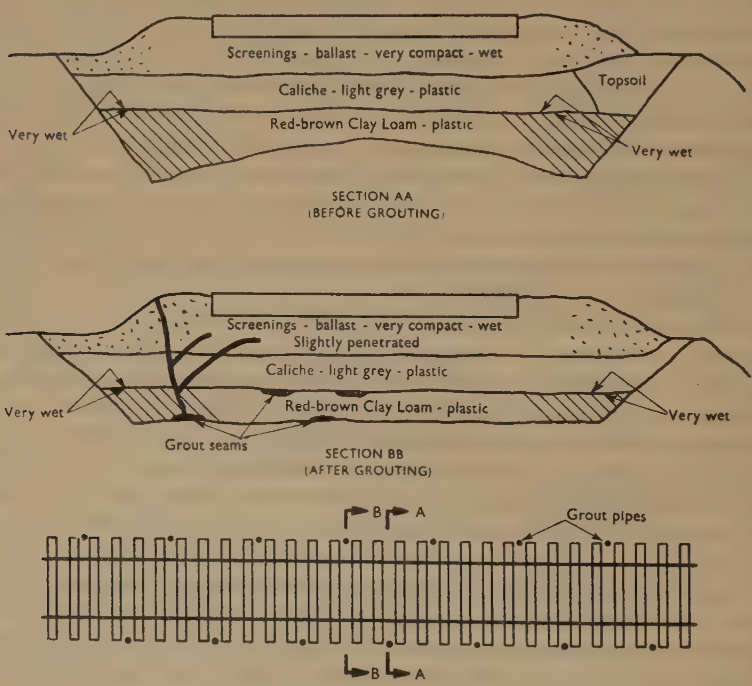
No wholly satisfactory reasons have been given for the effect of the grout. Most of the work has been done on embankments or on level ground with a raised formation. It seems doubtful if the method would be as effective in a cutting with the ballast choked with clay, where there is no egress for displaced clay slurry, other than into pipe drains alongside the track. Nevertheless, this method has proved itself to be of sufficient worth to warrant further trials in selected places.

Other methods of stabilization which have been used include sand-piling, poling boards driven on either side of the track, sand-filled camouflets, and sand-spreading on top of stone ballast. Varying lengths of

sleepers, up to 10 feet, may be used to increase the load-bearing area. Track blanketing is rarely done because this would be regarded, for accounting purposes, not as maintenance but as capital improvement subject to tax.

(6) *Points and crossings.*—As previously mentioned, sufficient space is usually available to enable standard lay-outs to be used and, furthermore, a relatively small number of standards is required. Since standard lay-outs are usually renewed like-for-like, it is possible to renew odd switch

Figs 8



Typical Track Grouting

rails, stock rails, crossings, etc., as they wear out without regard to the complete lay-out.

Standard crossings usually range from 1 in 8 to 1 in 20. The flattest turnout, a 1 in 20, gives a turnout radius of 52 chains and is normally restricted to a speed of 45 miles per hour.

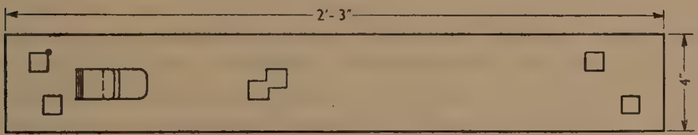
Main-line switch rails range in length from 16½ feet to 45 feet, and are planed either to fit against the side of the stock-rail head, or, in the case of the Samson point, to undercut the head. Various types of devices have been used to reduce wear on the switch tongue where the turnout track

is frequently used. One type, the switch point guard (*Fig. 9*, between pp. 276 and 277) is a manganese casting, fitted to the outside of the stock rail, which guides each wheel away from the switch tongue. This appears to be very effective for slow-moving yard tracks, but is not used on fast lines.

It has been found that the wheels of rolling stock develop a false flange on the outside of the tire which causes severe wear as it passes over the top of the stock rail. Some companies eliminate this by grooving the stock rail to take the false flange, whilst others are experimenting with flame hardening of the rail surface and heat-treated rail to increase resistance to wear.

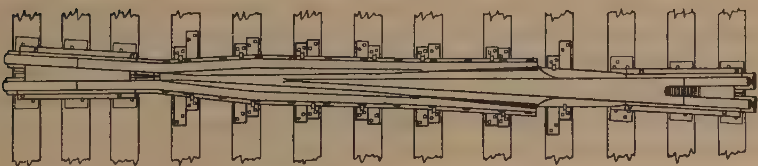
A type of adjustable brace to the stock rail is frequently used on

Figs 10



PLAN

ELEVATION
Scale: ⅛ full size
TWIN BASEPLATE



USE OF TWIN BASEPLATES

alternate sleepers to permit rail wear to be taken up and the switch adjusted to a good fit. To eliminate the necessity for a number of different-size baseplates between the last slide-plate of the switch and the first standard plate and at the crossing, twin half-plates are often used which can be adjusted to suit the varying distances between the rails (*Figs 10*).

A powdered graphite lubricant has been introduced for use on slide plates. At busy locations this lubricant has been found to be effective for 2 to 4 weeks and, at some locations with light traffic, for 6 weeks.

Three types of crossings are available :—

(*a*) The manganese insert type in which the insert and wing rails are cast in 12-per-cent manganese steel (*Fig. 11*, between pp. 276 and 277). The adjoinging rails are bolted to the outside of

the casting to form straps. This is the most commonly used type of crossing, giving good service under heavy traffic.

(*b*) The solid manganese cast type, which is a complete manganese casting joined to the adjacent rails by fishplates. This has the disadvantage that it tends to develop cracks and, having no rails to act as straps, it may fail suddenly; it is seldom used for acute crossings.

(*c*) The built-up rail crossing similar to that used in Great Britain, with the difference that there are continuous blocks from the end of the wing rails to the throat. This type of crossing has been used mainly in minor lines and small yards because it wears out rapidly. Its use has been revived in recent years, however, since it is found to give better service when the crossing is heat-treated before or after fabrication.

The relative costs of these three types of crossing are approximately (*a*) 1·2 : (*b*) 1·1 : (*c*) 1·0 (untreated). All three types of crossing can be built up by welding, the manganese castings by electric-arc welding only. It is the usual practice to carry out periodic repairs to crossings in track until they become badly worn, when they are replaced by new crossings. Old crossings may be repaired and rebuilt in the shops and then re-laid as second-hand material in branch lines or yards.

Check rails are fitted only opposite crossings, and generally a shorter check is used than in Great Britain. There are a number of different types of check rail in use, the most common being the one-piece manganese cast type which is made in 8-foot-6-inch, 10-foot-6-inch, and 12-foot lengths. Another common type is the rolled steel rail with a central clamp and the ends bolted through blocks to the running rail. Check rails are never extended back towards the switch on a sharp turnout, nor are continuous check rails required on curved plain line; thus the above standard-length check rails are adequate for all purposes.

Diamonds and slips may also be constructed with built-up rails, manganese inserts, or solid manganese castings for the obtuse crossing. Wide-angle intersections between the lines of different companies are frequently encountered and are difficult to maintain. The most common type is the solid manganese casting built up in sections (*Fig. 12*, between pp. 277 and 277).

One other type of crossing which is worthy of mention is the "self-guarded frog," an acute crossing requiring no check rails (*Fig. 13*, facing p. 277). It is a cast manganese crossing with raised lips on the wing rail portions of the casting which guide the wheels into the correct flangeway by pressure on the back of the wheels. These appear to be very effective and are widely used in yards by certain companies.

(7) *Curves.*—It is the practice in the United States to express curvature in degrees, by the angle subtended by a 100-foot chord of the curve at the

centre of the circle. Thus, a 1-degree curve (approximately 87 chains radius) is that curve on which a 100-foot chord subtends an angle of 1 degree at the centre. Various methods are used to ascertain the required cant on a curve, the general maximum being 6 inches with a 3-inch deficiency. Some companies fix the speed limits on curves purely by the riding qualities of a train. This is possibly more reasonable than an exact mathematical calculation, since modern coaches are fitted with soft springing with the result that the cant felt by the passenger is considerably less than that of the track. The use of flange oilers on sharp curves is fairly widespread, but lengths of severe curvature are frequently associated with heavy upgrades which may preclude the use of oilers, owing to loss of grip by the wheels.

Track Maintenance

During the past 15 years, maintenance practices have undergone radical changes as a result of the development of mechanical aids. Nowadays there is a machine for almost every maintenance operation, although some are of doubtful economical value.

(*a*) *Day-to-day minor maintenance.*—Essential day-to-day maintenance is carried out by length-gangs. The strength of these gangs varies considerably both seasonally and according to locality. The policy of companies governed by the winter weather is to employ only sufficient labour during the winter to carry out essential maintenance and to double or treble this force during the summer months for major maintenance and renewal work, by the employment of casual labour. A typical length-gang may consist of a ganger and two or three men for 10 track-miles of main line during the winter, and a ganger and five or six men during the summer. On lines with a high density of traffic, the use of petrol-driven trolleys has been found to be impractical and highly dangerous, so transportation of men and materials is normally by road vehicle. On less busy lines each gang is supplied with a trolley. Rules governing the operation of these trolleys are less restrictive than those on British Railways, and it is usually up to the ganger to make sure that the line is clear by telephoning control. From then on he may move freely on his own responsibility without the protection of hand-signalmen.

The general tendency is to keep the work of the length-gang to the minimum, or to dispense with them altogether and to undertake all major maintenance work by large highly mechanized extra gangs. Length-gangs are not generally mechanized, since this is neither practicable nor economical. Hand tools only are normally issued, although small compressors may be loaned for small tamping jobs. The duties of the length-gang are confined to inspection of the track and such minor work as may be necessary to keep it in good condition. Daily inspection of the track by the ganger or track walker is not regarded as essential except in heavily worked terminal areas or where formation conditions are known to be

troublesome; elsewhere a bi-weekly inspection is considered adequate, augmented by casual inspections by the gang, the inspector, or other supervisory personnel, in the normal course of their work. A rather different organization may operate in a large terminal area, where it is found to be economical to amalgamate two or more yard-gangs and to supply them with mechanical equipment.

The most common type of gang for major maintenance work is that equipped with a large-capacity compressor capable of operating sixteen hand tamping tools. This gang will set a length of track to line and level, if necessary making a small lift, without any interference to high-speed traffic. Experience has shown that the air tamping tool gives the best results and is the most reliable in operation. The tamper with the individual power unit is not generally favoured, even for small gangs.

On major maintenance or behind a ballast renewal operation, multiple tampers are used. There are several types of these in use employing different principles of vibration or force or a combination of both. Two types which appear to be fairly successful reproduce the action of the hand-operated air-tool (*Fig. 14*, facing p. 277, shows one of them). However, it is difficult to assess the relative merits of these tampers without considerable experience of their operation. When such tampers are used to make a general lift, they are preceded by a gang setting up the track to the required level by power- or hand-operated jacks and hand tamping at intervals.

No hand ballast riddling is carried out by the length-gang. All ballast cleaning except through points and crossings is done mechanically by an extra gang. All of the ballast cleaners seen were designed to clean the ballast in the cess or the six-foot, outside the sleeper ends. Separate machines are available to scrape the crib ballast into the cess and six-foot, whence it is picked up by the cleaner. It seems to be regarded as totally unnecessary to clean the ballast below the sleeper bottom, and normally the cribs are rarely cleaned. It is considered that, if the shoulders and six-foot way are kept clean, the natural flow of the water will remove most of the dirt from the cribs. When it is considered necessary to renew the ballast completely, a combination of the crib scraper and a type of bucket elevator can be used to remove the ballast. This, however, is rarely necessary because the track can normally be lifted on top of the old ballast, there being comparatively few overhead obstructions.

A type of ballast cleaner which appears to work satisfactorily is one which works clear of gauge either in the cess (*Fig. 15*, facing p. 292) or the six-foot. The cleaner is propelled along ratchet rails placed on the sleeper ends, and the speed of operation may be adjusted according to the proportion of dirt in the ballast.

A further type of machine which is widely used is the ballast-cleaning train (*Fig. 16*, facing p. 292). The train is equipped to clean on one side of the track only and to dispose of the dirt extracted by a series of belt

conveyors into hopper wagons, where disposal at the track-side is not possible. On the side of the track not being cleaned there is no obstruction at any time. On the side being cleaned the bucket elevators can be retracted to clear gauge. The train may seem to be rather elaborate, but it can attain a cleaning speed of $\frac{1}{2}$ mile per hour, with no speed restrictions before or after the operation, which makes the very most of available possessions. As a general rule, the train is worked 24 hours a day during the summer months, working busy daylight hours in multi-track territory, with the necessary traffic alterations, and at night in double-track territory with single-line traffic. It is considered economical to move the train up to 25 miles between day and night operations. There is another make of ballast-cleaning train, not seen by the Authors, which can clean both sides of the track at the same time.

Other maintenance operations which may be carried out by extra gangs are weed-killing, grass-cutting, fence-repairs, and maintenance of drains. As already mentioned, these gangs may be transported either by rail trolley or, preferably, by road. Normally the railway right-of-way is sufficiently wide to allow access to most parts of the track by lorry. In certain ideal cases a dirt service road has been formed alongside the track, but this is rarely possible in built-up areas.

(*b*) *Track renewal.*—It is not the general practice to renew track "out-of-face" (that is, rails, sleepers, fastenings, and ballast complete). The normal practice is to renew the rail and fastenings at one time, to spot-renew sleepers on an annual programme, and to renew ballast rarely.

Ballast renewal is normally only carried out when it is desired to replace an inferior type such as pit-run gravel; where a substantial lift is not possible; or where there is a bad formation. Mechanical appliances used for this work have already been mentioned.

Sleepers which require renewal are marked by the inspector when preparing his annual programme. They may be renewed by the length-gang as a part of normal maintenance or by an extra gang equipped with mechanical equipment which has been developed for pulling and inserting sleepers.

Rail renewal is usually carried out on a fully mechanized production-line basis. Normally a gang will work a day or two on one rail, and then return to deal with the other rail. A reasonable output in an 8-hour day, with full possession of the track, including travelling time to and from the site (usually from dormitory coaches), would be $2\frac{1}{2}$ to 4 miles of single rail. Naturally this gang can only operate at maximum efficiency when possession of the line can be obtained, although it is possible to work the gang between trains, without speed restriction, the efficiency depending on the frequency of trains. When it is desired to clear the section, for the passage of a train, the rail gap is closed up and all light machines are lifted into the ditch alongside by the crane. A $6\frac{1}{2}$-ton diesel crane is the type most frequently used; this can travel at more than 30 miles per hour under its

own power and can therefore run quickly to the nearest siding, or can alternatively be put off at the line-side by building a set-off, but this is rarely necessary.

The typical procedure of a large rail-gang is as follows (new material has previously been laid at the line-side, and material removed from the track will be picked up at some convenient later time) :—

(1) Old spikes are pulled out by four machines, each with a crew of three.
(2) Spikes missed by the machines are pulled by hand.
(3) Fishbolts are removed by power wrenches.
(4) Crane lifts out old rails. Alternatively, rails may be barred out by a specially shaped bar.
(5) Old baseplates are removed by hand and creosoted plugs are rammed into the old spike holes.
(6) Tops of sleepers are adzed to a level by four rotary power-adzing machines. Before adzing it is necessary to clear any ballast away from the sleeper tops to avoid damage to the blades.
(7) Creosote is sprayed on to the adzed surfaces.
(8) New baseplates are positioned, either by hand or by one of the machines which have recently been devised to set the plates to gauge.
(9) Crane lays new rail. All new rails have previously had a pair of fishplates hung on one end and the adjacent rails are slotted into them. This enables the crane which is laying rail to move forward immediately.
(10) Joint bolts and packings, if used, are fixed and tightened by power wrenches. If packings are not used, rail ends are greased.
(11) Gauging gangs set the track to line, and hand-spike at intervals.
(12) Sleepers may be bored to take spikes. This practice varies and depends on the size of baseplate previously fitted to the sleeper.
(13) The remaining spikes are driven by pneumatic spike-hammers.
(14) Track anchors are fixed by hand.
(15) Rail ends are hardened where this has to be done on the site, and may be cross-ground to prevent flaking.
(16) As a final operation, rail bonds are fixed.

The total manning of such a gang is about 120 to 150, of which about 75 per cent may be unskilled casual labour. A rail-laying gang of this size obviously takes some time to get strung out, the distance from head to tail being about 1 mile. If the line has to be cleared to pass trains, a smaller gang would have to be used, working more closely together.

(c) *Maintenance of mechanical equipment.*—It will be obvious that, if mechanical equipment is to be used effectively on a large scale, there must

be a thoroughly efficient maintenance organization. In one of the best systems, all plant is nominally controlled at head office; certain small items of plant are retained more or less permanently on district establishment, whilst the larger items of plant are subject to direct control by head office. There is a well-equipped central workshop which has three functions: (i) to carry out major overhauls to large machines; (ii) to inspect and repair machines and equipment returned from districts and to store them until they are required again (this refers mainly to items of seasonal use such as weed-sprayers, grass-cutters, and equipment used on summer maintenance programmes); (iii) to maintain records of all inspections and repairs carried out on the plant, both by the central workshop and the district organization.

Each district is responsible for the day-to-day maintenance of all plant issued to it and for the complete maintenance of smaller items. The district has a small workshop in which to do these repairs and one or two mobile fitters who inspect and maintain plant while it is in use. These fitters use a small van equipped with bench, vice, etc., and a selection of the smaller spares most frequently required.

For each type of machine, there are operating and maintenance instructions and new operators are tested on these instructions by travelling inspectors from head office. These inspectors are responsible for general efficiency in operation and maintenance, and in addition help to formulate the central workshop programme of heavy maintenance in advance. A monthly list of plant is circulated to those concerned showing the location and work on which each item of plant is engaged. This is made up from periodic returns from the district, the central workshop, and from the operator direct. Every effort is made to ensure that plant is being used economically and that maintenance is being carried out efficiently to a programme.

Freight Classification Yards

During the years since the 1939–45 war there has been a substantial increase in the modernization of classification yards and, in most cases, it has been possible to combine the work of several yards into one mechanized yard. The normal yard consists of three sections, namely, reception, classification, and departure. With the great length of the average goods train it is generally necessary to make up a train for departure from two or three cuts in the classification yard. Two types of retarders are in general use, the electric retarder operated by a motor-and-lever system, and the electro-pneumatic retarder operated by an air-cylinder-and-lever system (*Fig. 17*, between pp. 292 and 293). In most yards there are three stages of retardation—primary before the king switches (used mainly for cuts of several heavy wagons), secondary, and tertiary. This retardation ensures the minimum of damage, provided that correct humping speeds are used and that the gradients throughout the classification yard

are well maintained. One of the latest retarder developments is in the use of automatic speed-control, whereby the operator selects the speed appropriate to the weight and number of wagons in the cut and subsequent control of retardation to achieve this speed is fully automatic. In this particular yard one operator controls five retarders and switch movements to twenty-five tracks in the classification yard on an automatic route-selection board.

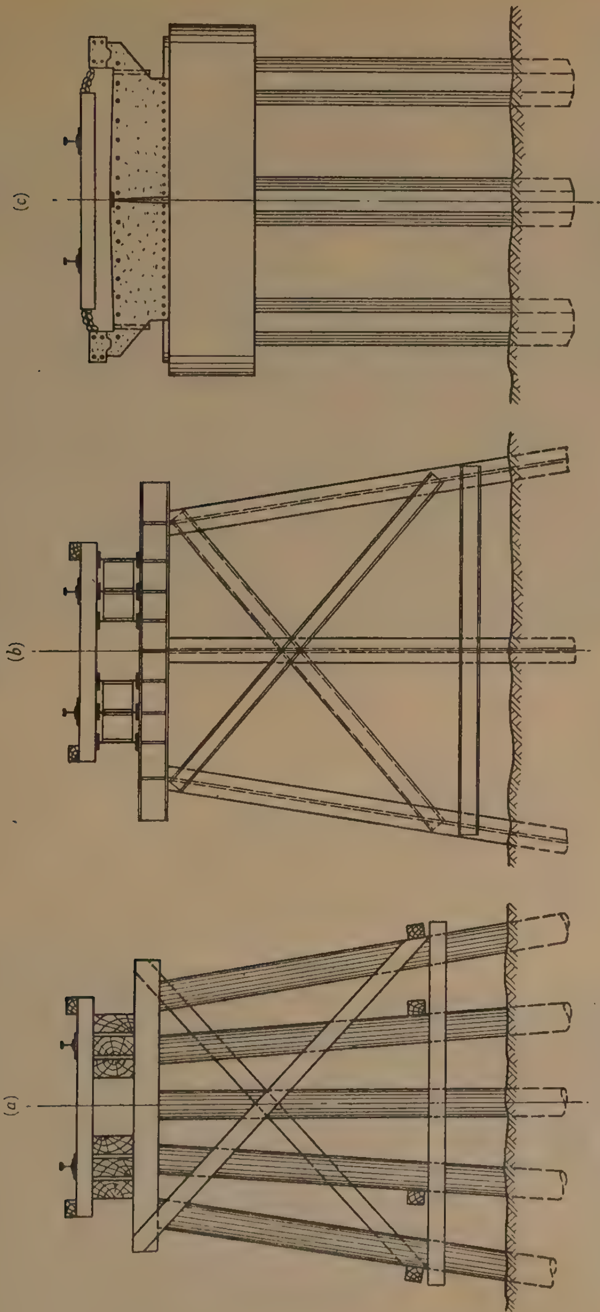
Two other features are worthy of mention. First, lighting at the necks of the yard is normally very effective from batteries of lights on 100-foot steel masts. Secondly, wide use is made of radio, talk-back speakers, and other inter-communication systems throughout the yard, which allow a very close control to be kept over all yard operations.

Bridges

It has been estimated by the A.A.R. that the track mileage of bridge construction in the United States is divided approximately in the following proportions: 40 per cent timber, 40 per cent steel, and 20 per cent concrete and masonry. From these figures it is evident that timber still plays a considerable part as a material for railway bridges. The usual form of construction is the pile trestle with timber beams (*Fig. 18* (*a*)) and either an open deck, with rails secured direct to the transverse bridge-timbers or a close timber deck for use with ballasted track. The latter type of deck is preferred because it is easier to maintain and there is less risk of fire. Water-barrel platforms are provided, and on long bridges timber draught-curtains are built at 150-foot intervals. The fire hazard is a serious problem on this type of bridge, and the A.A.R. has been conducting tests on the fireproofing of bridge timbers. Walkways and handrails are only provided where necessary—for example, where train crews have to dismount to operate switches. Generally the spans, spacing of piles, and other details are standardized, so that stocks of piles, beams, bracings, and fastenings can be kept to hand for use in emergency. The piles are usually driven by on-track steam-operated pile-drivers, and when a bridge is reconstructed, the new pile trestles are placed between the existing bents, thus enabling the pile-driver to operate on the old structure. The height of this form of construction is normally limited to 30 feet. For higher bridges, braced steel tower trestles are used to support steel girders, although many of the railways in the west have framed timber bents as high as 60 feet or more. The estimated life of treated timber piles subjected to periods of immersion in water is from 45 to 50 years. On one railway in the west it is the practice for the Works Inspector to inspect all timber bridges on his district every 6 months, and in addition the District Engineer is required to carry out his inspection annually.

The traditional trestle form of construction has been adapted for use with reinforced concrete and tubular or H-section steel piles (*Figs 18* and *19*). The piles are usually capped with concrete, and the superstructure

Figs 18



(a) 15'-0" -SPAN TIMBER TRESTLE

(b) 20'-0" -SPAN STEEL TRESTLE

(c) 18'-0" -SPAN REINFORCED-CONCRETE TRESTLE

TYPICAL PILE-TRESTLE BRIDGES

consists of either reinforced-concrete slabs or wide-flange rolled steel beams. These beams are available in sections up to 3 feet deep with a maximum weight of 300 lb. per linear foot, and are extensively used for small-span bridges. The decks of these bridges consist of either open or closed timbers placed directly on the steel beams, or precast or cast-in-situ reinforced concrete. Where a waterproof decking is required—for example, over a road—wrought-iron floor plates are used to offer greater resistance to corrosion. The joints are covered with wrought-iron strips welded to the main plates. Expansion joints are provided where necessary in the form of U-shaped copper strips brazed to the wrought-iron plates. The floor plates are then covered with a five-ply membrane consisting of an asphalt primer, asphalt top coat, waterproofing felt, and saturated cotton fabric, topped with asphalt plyboard consisting of hard bitumen fibreboard impregnated with bitumen.

As a direct result of the experience of the costly and difficult repair work which is now necessary on some existing concrete bridges constructed between 1920 and 1930, reinforced concrete is not widely favoured. It is avoided by at least one railway in the west for new superstructures in underbridges, except in cases where architectural features are required, and then it is restricted to face girders. In this connexion, the A.A.R. has recently commenced a survey of existing concrete structures to study the causes of deterioration of the concrete and the nature and methods of carrying out repairs. The need for prestressed concrete as a bridging material on American railways has not arisen since steel and timber are more readily available. However, the Portland Cement Association has tested a prestressed concrete slab, designed for railway loading, to show the possibilities of the material.

Plate girders for railway bridges are almost entirely of riveted construction. A typical modern intersection bridge is shown in *Fig. 20*. Doubts are still expressed with regard to the fatigue strength of welded girders. Tests have been carried out in connexion with the strengthening of cross-girders by the addition of welded plates, and valuable information has been obtained about the type of details to avoid from the point of view of fatigue strength. Further tests are in progress in the field of cumulative fatigue, using devices fitted to fatigue-testing machines which apply stress range patterns designed to reproduce those found in practice.

A most important development in steelwork construction is the introduction of high-strength bolts. There are two main uses: (*a*) in connexions subjected to repeated dynamic loads and vibrations, where rivets are continually working loose; and (*b*) in field connexions where, normally, the temporary bolts used during erection are replaced by field-driven rivets. Laboratory tests and field installations on several railway bridges have shown conclusively the superiority of high-strength bolts over rivets under repeated loads. One difficulty still to be overcome is a practical method of applying the required bolt tension. Torque wrenches are not

Fig. 15

The Mole Ballast Cleaner

Fig. 16

The Brownhoist Ballast-Cleaning Train

Fig. 17

North Platte Yard (U.P.)

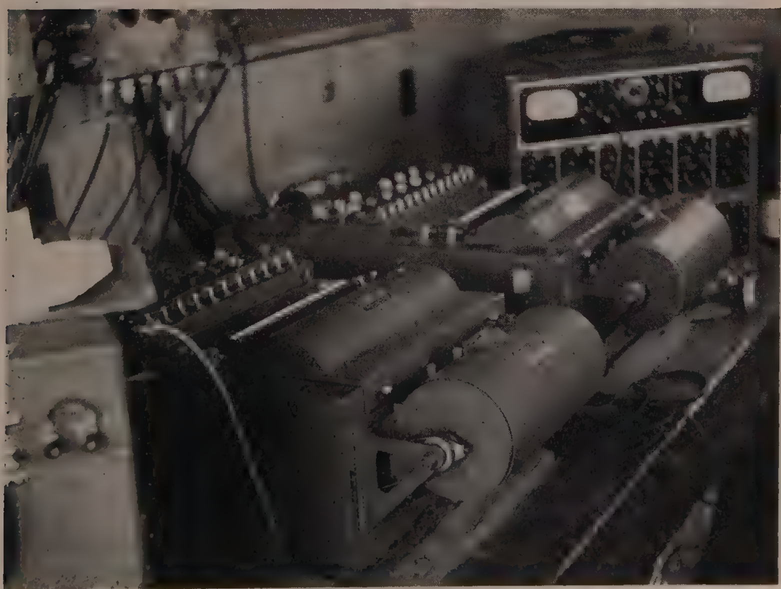
Fig. 19

Tubular Steel Trestle

Fig. 20

Modern Intersection Bridge, Morrisville, Pa. (P.R.R.)

Fig. 21



Interior of A.A.R. Mobile Laboratory, showing Bridge Test Equipment

Fig. 22

DIESEL MAINTENANCE SHOP, HARRISBURG, PA. (P.R.R.)

Fig. 23

UNION STATION, OMAHA, NEBRASKA (U.P.)

entirely satisfactory for this purpose, but it is hoped to show that the bolts may be safely tensioned into the plastic range of the steel.

It is common practice in through-type girder bridges to stiffen the top flanges by brackets with web plates attached to the cross-girders. Such brackets are susceptible to damage in the event of a derailment, as was evident from at least two such bridges. Guard rails are provided approximately 1 foot inside the running rails, with the object of preventing a derailed vehicle from leaving the bridge or damaging the main girders in through-bridges.

An interesting item of bridge repair work, carried out by contract, involved an unusual method of placing concrete. The bridge consisted of longitudinal steel beams supporting reinforced-concrete slabs which had deteriorated badly. It was decided to encase the beams in concrete, without interference to traffic. Shuttering was placed under the beams and washed gravel aggregate was blown into place from below. Grout was then injected under pressure at several points between each pair of beams. When grouting was complete in each section, external vibrators were applied.

Perhaps the most outstanding single item of bridge reconstruction observed was the bridge carrying a single-line railway over the Ohio River. The work consisted of the renewal of the nine long truss spans over the river proper. The two 518-foot spans and four of the 400-foot spans were replaced with through truss spans of the same lengths. The remaining three spans were replaced with six deck truss spans of 197 feet, with additional piers midway between the existing piers. The new trusses were erected one at a time on falsework alongside the existing bridge, the short-span trusses being utilized for this purpose. The old trusses were rolled out on to sledges, from which they were launched on to the river-bed 130 feet below. The work of salvaging the scrap metal was carried out at low water from barges. In the new trusses, plates with manholes cut at frequent intervals have been used instead of lacing bars between the components of the main members. This form of construction reduces local distortion and is, of course, easier to paint. Inspection walkways have been provided throughout the length of the top chords of the trusses.

Bridge Inspection and Maintenance

The District Engineer is generally responsible for the inspection of bridges and structures, the work being done by Bridge and Building Inspectors or in some cases by the District Engineer himself. Reports of condition and necessary repairs are sent on standard forms to the Bridge Engineer at head office who makes up an annual programme for painting and repairs and submits a budget to the Chief Engineer and thence to the Vice-President (Operations).

A number of steelwork and concrete crews are kept working throughout

the year carrying out repairs to bridges, structures, and tunnels. The work is planned for the whole system by the Bridge Engineer, but each District Engineer is responsible for the supervision of the work in his district. The repair crews are very well equipped—for example, the steel-work crews are equipped with cranes and air compressors and include riveting and painting gangs. The crews are housed in work-trains which convey the complete outfits to the various jobs throughout the railway system.

Loads, Stresses, and Research

Most main-line underbridges in the United States are designed to carry Cooper's E 72 loading, although E 75 is sometimes used. Cooper's E 72 loading consists of two 2-8-0 locomotives having static driving-axle weights of 72,000 lb. (32 tons), followed by a load of 7,200 lb. (3·2 tons) per linear foot. Impact forces are allowed for, lurching being taken as 10 per cent of the static axle loads, whilst the effect of hammer-blow and track irregularities is given by a set of impact formulae, depending on the span range and type of construction. The complete design specification is included in the A.R.E.A. Manual.

The A.A.R. has recently completed a study of impact stresses on small-span underbridges and the results have been incorporated in the A.R.E.A. specification for steel railway bridges. There is an appreciable reduction in the total impact effect compared with that previously used in design. A similar programme of tests has been carried out on medium-span girder bridges and the results will be published shortly.

The structural division of the A.A.R. Central Research Laboratory consists of a field staff which carries out tests, and an office staff which analyses the data and prepares reports for publication. When a particular site is accessible by road, the equipment is transported by lorry fitted out as a mobile laboratory (*Fig. 21*, between pp. 292 and 293). When access by road is not possible, the equipment is sent by rail and housed in a sectional hut.

Electric wire-resistance or electro-magnetic strain gauges forming one leg of bridge circuits are used to measure the strains. The unbalanced voltage from the bridge circuits is amplified before being fed to the oscillograph galvanometers. Two oscillographs are used, each with twelve galvanometers fitted with small mirrors which reflect beams of light on to a moving reel of photographic paper. A phase discriminator is included in the electrical circuit, thus enabling tensile and compressive strains to be recorded. Trip switches are attached to the rails at a given distance apart and, when operated, a record is made on the photographic paper, thus enabling the position of a locomotive and train to be related to the main stresses. A time base is also recorded on the film so that the speed of the train can be calculated accurately. An approximate value of the speed is noted by means of a stop watch.

From the recorded strains it is possible to determine the effect of lurching, hammer-blow, and the total impact. For the purpose of design, formulae have been derived to include all impact effects (except lurching, which is considered separately), expressing the percentage of the axle loads to be taken as impact in terms of the span, for equipment with and without hammer-blow.

Miscellaneous Installations and Facilities

It is not possible to discuss in detail all the works with which the civil engineer is concerned, but it is considered that the following brief notes may be of interest.

(*a*) *Motive power depots.*—With the recent swift transition from steam to Diesel-electric power, few modernization schemes have been carried out on engine sheds, coaling plants, ash pits, etc. On the other hand, large numbers of Diesel shops have been built which provide excellent facilities for inspection and maintenance. Platforms are arranged to allow inspection at three or four levels—usually inspection pit, axle level, cab level, and possibly roof level (*Fig. 22*, facing p. 293). Lift bridges are provided between platform ends to allow parts to be distributed by truck. Outside the shop the servicing facilities are arranged in sequence: fuelling, sanding, watering, etc., followed by washing; the locomotive then proceeds to the shop for inspection. Additional bays fitted with a drop-table for removing axles and motors are provided for units requiring heavier maintenance.

(*b*) *Stations* (*Fig. 23*, facing p. 293).—In the larger cities, station buildings are of elaborate design and are normally very pleasing architecturally, but it is difficult to see how they have been justified economically. As a result of the reduction in steam power and of the strict smoke abatement laws in force in most major cities, it is becoming increasingly possible to keep such buildings in a clean condition.

(*c*) *Goods sheds.*—In most large goods sheds, mechanical handling equipment has been introduced. Normally, unless traffic is fairly constant, conveyor-belt systems are not favoured. Most mechanization takes the form of electric or petrol trucks for towing carts, and fork-lift trucks for pallet working. However, several sheds have been equipped with systems incorporating belt-and-chain conveyors, powered rollers, etc.

(*d*) *Carriage cleaning facilities.*—Most coach yards are equipped with some type of mechanical cleaner. The most widely used type utilizes rotating brushes and consists of three operations: (i) a wetting spray; (ii) the cleaning operation in which a proprietary brand of cleaner is brushed in by powered brushes; and (iii) the rinsing, in which dirt and cleansing agent are washed off by clean water brushed on by rotating brushes. This gives a very clean finish to both coachwork and windows, particularly on modern rolling stock; on older stock it may be necessary to employ hand cleaners with brushes to clean awkward corners.

(e) *Switch heaters.*—In winter, the problem of keeping switches clear of snow and ice is an important one, as it is in Great Britain. It is generally agreed that the electric heater is the most efficient and requires least attention where an adequate electrical supply is already available, as in retarder yards. Several types of gas installations have been evolved, utilizing either piped gas where available or propane or butane gas in liquid form. There are two disadvantages of this type of heater: first, they are difficult to light by remote control; and secondly, they are liable to be blown out by a strong gust of wind or by a train passing at speed. However, these are not serious drawbacks where a man can be in attendance. A third, and simpler, type of switch heater in common use is the kerosene pot, which is simply a paraffin lamp that is placed under the switch rails in a hole dug in the ballast, three to five pots being required to each switch rail. These are quite efficient but tend to get dirty, require fuelling frequently and are liable to be blown out.

Observations on American Practice

In making comparisons between practices in Great Britain and those in the United States, a number of differences in conditions should be borne in mind. These are:—

(1) The different relationship between wages and the cost of materials. Material costs are roughly equal, at the prevailing rate of exchange, whereas American wages are three to four times those earned in Britain. This makes the substitution of manual labour by machines in Britain more difficult to justify from an economic point of view.

(2) Axle loads of locomotives and rolling stock are, in general, higher in the United States than in Britain, and thus require heavier track and bridge construction.

(3) Springing of rolling stock, particularly that of the goods wagon with the long wheel-base and the four-wheel bogie at each end, appears to be constructed to negotiate what, in Britain, would be called rough or even dangerous track.

(4) The width of the average railway right-of-way facilitates the access of road vehicles and off-track machinery to most parts of the track.

(5) In many parts of the United States, traffic density is light compared to that on British Railways, and maintenance and renewal work can consequently be carried out under the best conditions. Nevertheless, there are many stretches of track in the industrial areas of the east where traffic densities are comparable to those on the main lines of Britain.

(6) The average individual in the United States usually has a good understanding of the internal combustion engine, probably

because he usually owns a car. Thus, even so-called unskilled labour is able to operate mechanical equipment intelligently without a great deal of additional instruction, which is of considerable benefit.

(7) The availability of different materials affects the design of structures and has already been mentioned.

These differences in conditions must be considered, since they frequently make a direct comparison difficult or even impossible. Nevertheless, they should not be used as an automatic reason for the outright rejection of American practices as applicable to the railways of Britain. The Authors wish to suggest that the following may be worthy of consideration in the effort to adapt practices to changing techniques and economic conditions.

(*a*) *Track construction.*—The practice of allowing free movement between rail base and baseplate has been mentioned, and it may be that this would tend to reduce movement of the sleepers in the ballast, thus reducing the work necessary to keep the track to level. The practice of staggering rail joints would appear to have some merit, although it is doubtful that this should be by as much as half a rail length ; a difference of a few feet might be sufficient to reduce wear on joints and rolling stock, without excessive lurching.

Cast manganese-steel crossings cost very little more than built-up carbon-steel crossings in the United States, whereas the former have a very much extended life with few bolts requiring maintenance. If this price relationship can be approached in Great Britain, and there is no apparent reason why it cannot be, a wider use of this type of crossing might prove economical.

The use of long lengths of welded rail in the United States has passed the experimental stage on many companies. There would appear to be ample justification for extended tests in Britain. It is considered, in the United States, that high installation costs are more than offset by reduced maintenance costs, both for track and rolling stock, and by better riding conditions. No difficulties are experienced with temperature variations, in continuous lengths from 2 to 3 miles, provided that the track is well anchored and that certain other precautions are observed.

In normal jointed track the end-hardening of rails is becoming standard practice in the United States. This would appear to be economically justified where end batter is abnormal, particularly on electrified track.

(*b*) *Track maintenance.*—Mechanization of maintenance is at present increasing in Britain. The use of "area" gangs, fully mechanized, appears to give the most economical organization. It is argued that this type of organization tends to destroy "pride of length," but there is no reason why this should not be replaced, to some extent, by "pride of area." However, it may be that the percentage of supervision will have

to be increased, either by reduction of the size of Inspectors' districts, or by the appointment of Sub-Inspectors. This would permit a much closer supervision, so that gangs ceased to work by an annual calendar and would work instead to a detailed programme compiled by the Inspector as a result of frequent inspections. Closer supervision by the Inspector would enable him to exercise a better control over the gangers, and thereby train them in correct maintenance practices.

As the percentage of flat-bottom track in running lines increases, it should be possible to reduce regular track inspection to twice a week, except where abnormal conditions exist, since very little serious deterioration can take place in a length of flat-bottom track between inspections. Periods of inspection would have to be increased in abnormally hot weather unless a greater use was made of rail anchors. Power-spray lubrication of joints and supersonic inspection of joint areas would reduce the time required for joint maintenance and might enable the oiling of joints to be carried out more than once a year, if thought necessary.

(*c*) *Track renewal.*—The policy of partial renewal of track has practically disappeared in Britain. However, it is questionable, whether, in all cases, partial renewal is necessarily uneconomical compared with complete renewal under the pre-assembled track systems. The complete renewal of plain line before sleepers and ballast are worn out involves the handling and grading of enormous quantities of material and, where second-hand renewals are less than new renewals, there is a surplus of materials which has to be sold, although still serviceable. The American policy of partial renewal has certain advantages, in that possessions and speed restrictions are reduced—for example, 2 miles of rail renewal requires an 8-hour possession only, and no speed restriction, whereas 1 mile of complete pre-assembled track renewal requires a 12-hour possession of two lines and 7 days restriction. Depending on train frequency, it may be possible to renew some rail between trains.

Spot renewals of sleepers can be done by an area gang without interference to traffic, and sleepers need only be renewed when their useful life is finished. This is of considerable benefit because it is considered that the handling of a sleeper from one location to another automatically reduces its life by up to 5 years. By spot renewal of sleepers in this manner, the average standard of sleeper in the track is practically constant, permitting the use of each sleeper to the full extent of its life. A policy of partial renewal such as this would necessitate a more extensive mechanical ballast-cleaning programme.

(*d*) *Possessions.*—One of the main factors affecting the use of mechanical equipment for maintenance and renewal is possession of the track. It is the duty of the Engineer to reduce track possessions to the minimum by carrying out as much work as possible under traffic—by the use, so far as possible, of off-track machinery and by the detailed planning of every job involving a possession, thereby reducing traffic delays. In the United

States. many lines are signalled in both directions; facing crossovers are placed at convenient points for single-line working and, by a great extension of radio and other communication between signal box, train crews, and engineering gangs, it has been possible to increase the flexibility of traffic movements and so reduce delays.

(*e*) *Bridges*.—In view of the increasing use of reinforced concrete on British Railways, careful attention should be paid to any conclusions published by the A.A.R. or A.R.E.A. as a result of the survey now in progress of deterioration of concrete structures.

The impact formulae for design derived by the A.A.R. from the results of their tests and included in the A.R.E.A. Manual are referred to in the proposed new B.S.S. bridge specification. A study of the A.A.R.'s bridge-testing equipment and the method of carrying out the tests and analysing the results might prove useful if a bridge-testing programme is contemplated in Great Britain.

Relatively little is yet known about the effects of fatigue on welded girder construction. Perhaps the work on cumulative fatigue now being conducted at the University of Illinois will throw some light on this vital matter. Particular attention should be paid to diaphragms or bracing between deck girders, because these members may be subject to reversals of stress, which is one of the worst conditions from the point of view of fatigue.

If and when steel becomes more plentiful, it may be possible for large broad-flange beam sections, similar to those available in the United States, to be produced in Great Britain. Such sections would be of great value in small-span railway bridges where a deck construction is possible. The introduction of high-strength bolts would be another asset to the Bridge Engineer for site connexions and in joints where rivets are continually working loose.

(*f*) *Technical instructions*.—In the United States, technical standards and recommended practices are contained in the A.R.E.A. Manual. In addition, each company issues a maintenance-of-way rule book, which contains all instructions required by inspectors and gangers for track and structure maintenance. Changes in policies and practices are brought into being by issuing amendments to the rule book. Thus, ideally, an inspector or ganger has only one publication to consult.

In Great Britain, certain books of instructions are in existence, but, in general, instructions are issued by circular letter. These are easily lost or damaged and, not being part of a general instruction book, may contradict previous instructions without countermanding them. It is thought that the work necessary to produce such a handbook of maintenance practice would be justified. Supervisors would be fully aware of what is required of them without having to refer to a pile of instruction letters.

(*g*) *Safety*.—Strenuous efforts are made throughout industry to promote safe working of employees, and it is considered that more could be

done in this direction on British Railways. The importance of safety is continually brought to the attention of employees by a number of devices such as daily reading of a safety rule by the ganger, posters, films, lectures, and various types of competitions with prizes for sections having the best safety record. This safety drive should apply not only to staff engaged on work on the track but also to workshop staff where the enforcement of safety regulations, such as the wearing of protective equipment, is of paramount importance.

(*h*) *Development of mechanical equipment.*—There is, on British Railways, a large market for equipment, of the right type, to facilitate maintenance. It is considered that greater efforts might be made to convince private firms of this potential market and to encourage them to develop the plant required by the railways.

Conclusion

The Authors have attempted to present a factual, rather than a critical, survey of American practices with particular emphasis on the more recent advances. It is thought that a study of these developments might indicate possible improvements in railway construction and maintenance in Great Britain, particularly in connexion with flat-bottom track, which has been standard in the United States for many years.

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The Paper is accompanied by sixteen photographs and five sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Authors, introducing the Paper, remarked that there were two non-technical aspects of their year's visit to the United States which they wished to mention as being characteristic of the average American engineer and which could be more widely imitated in Britain.

First, there was the "commercial outlook" of the American railway engineer. It had often been said, usually in a derogatory manner, that an American would do anything to turn an honest "buck." That might have an adverse effect on his character in many ways, but it did produce an engineer who was a commercial part of a commercial undertaking. It was not suggested that American engineers were ideally economical in all their working, but it was suggested that that attitude was a decided asset. Many engineers in Britain took the attitude that the commercial side of the undertaking was none of their business and that, by professional right, they were entitled to a certain minimum amount of money for the job regardless of the financial health of the concern. Others believed that expenditure was only a charge against a certain code in the budget and was of small consequence. It was considered that an attitude of mind should be cultivated which regarded every expense, necessary or unnecessary, as the carriage of so many extra passengers or so much extra freight from A to B.

Secondly, the Authors mentioned "enthusiasm," remarking that they might be considered naïve or even old-fashioned to talk about enthusiasm in times when there existed, not only in railway service, a "couldn't-care-less" attitude, and a false legend of always winning the last battle no matter how slow in the opening phases. They had been very impressed by the apparent genuine enthusiasm of American engineers, which might have a less desirable aspect in the excessive use of superlatives, but could only have a good effect on the working of the concern.

It was suggested that unless a spirit of commercial enthusiasm could be developed or acquired, railway engineers might find themselves working as drudges for a redundant asset.

The Authors concluded their introduction by showing a number of slides.

Mr A. H. Cantrell said that there was a tendency in the United Kingdom nowadays for railway engineers to think that as they and their predecessors had been railway civil engineers for so many years there was very little for them to learn from other countries. Whilst that might be true to a certain extent, they should nevertheless carefully consider methods in use abroad.

Conditions in Britain, however, were vastly different from conditions in the United States of America. The track occupations there were very different from those in Britain, and the facilities for getting alongside the track were much better. Therefore, although some of their methods

might be put to good use in Britain, it did not necessarily follow that they were always possible.

The method mentioned in the Paper of spreading oil on the fishing surfaces of the plates in the joints by pressure lubrication was not entirely satisfactory. On British Railways the plates were removed and the opportunity was taken of examining the joints. Since, in the United States, rail failures were quite common, and since it was mentioned in the Paper that the special rail-testing cars were not very efficient in picking up faults at the joints, Mr Cantrell considered that that was all the more reason for taking off the plates and having a look at the joints.

Mr Cantrell referred to the fact that in America they did not use reinforced-concrete sleepers, and said it would be useful if the Authors could give any reasons for the Americans not adopting them.

It was rather surprising to see that in America they also had trouble with pumping sleepers. In the United States the rails were fixed to leave a breathing space between the rail and the sleepers, so that the rails went up and down, but the sleepers not quite so much. The sleepers must of course move, but it was rather surprising that pumping sleepers gave trouble in certain places.

Figs 8 on p. 282 showed how pressure grouting on the formation was carried out and its effects. Mr Cantrell could not quite understand why in Section AA of *Figs 8* there was a curved surface at the bottom, but after pressure grouting had been carried out the surface became straight. Pressure grouting in formation should be embarked upon only after very careful consideration. In America it had been used quite a lot on embankments, and it was usually said that the method was equally successful in cuttings, but that, of course, meant only in very selected cuttings, because, unless the clay was fissured, it would be impossible to insert grout.

Mr J. Taylor Thompson said that certain of the observations on p. 296 were very interesting, because they seemed to imply that productivity in the United States, so far as manufactured articles were concerned, was about three or four times greater than in the United Kingdom. Presumably that meant that the wage-earners in America lived at a standard which, so far as those products were concerned, was three or four times the standard enjoyed in the United Kingdom. That was a very interesting sidelight on the American wages situation. It was important to realize that the high wages in America made mechanization more justifiable than in the United Kingdom.

Paragraph (4), p. 296, about the width of the average railway right-of-way seemed important. On the American lines, where the traffic density was not normally so heavy as in Britain, presumably on-track machines were quite easily used, whereas in Britain with its heavy traffic density, there was greater scope for off-track machines, which seemed to suggest that in developing new lines or altering old ones the path at the side of

the track should be widened wherever possible so as to make it practicable to use off-track machines.

The reference on p. 297, in the paragraph dealing with track construction, to free movement between rail base and baseplate was an interesting difference from British practice—a free movement of rail compared with a free movement of rail plus sleeper. In one case the impact was on the sleeper and presumably the damage was done there, and in the other case the impact was on the sleeper bed. Did that mean that the Americans had decided that it was better to have work to do on the sleeper rather than on the packing under the sleeper ?

With regard to staggered rail joints, mentioned on the same page, Mr Taylor Thompson said he had tried staggered joints on one occasion with the joint midway, and he had come to the conclusion that a staggered joint at half-rail length led to a heavy rolling action, whereas the existing joint produced a plunging action, so that something between the two was perhaps right, namely, a slightly staggered rather than a fully staggered joint.

More information about the Americans' considerable experience with long welded rails would be welcome. What maintenance savings had resulted from the use of long-welded track compared with non-welded track, and had there been any trouble with buckling or other long-welded rail difficulties ?

On the question of track maintenance, which was dealt with on pp. 297 and 298, it seemed that the Americans were following the same general lines of development as was Britain and the Continent ; that was to say, day-to-day track maintenance was becoming a matter for a light length-gang (an inspection gang) with a working gang operating to a detailed programme compiled by an inspector.

As already mentioned by Mr Cantrell, the power-spraying of joints was not necessarily in lieu of inspection. To what extent would the power-spray force lubricant into the fishing table of the rail ? Was it a case of injecting oil, or merely of covering the joint with oil externally?

The American policy of partial renewal was worthy of detailed consideration in Britain, where there had been for many years a tendency towards complete renewal, perhaps partly because of the convenience of prefabricated methods. But since timber sleepers had a variable life, there was a logical case for replacing them individually rather than by complete replacement of a long length. In the case of rails, the life for a long length was fairly uniform, but with timber, which was a natural product, there were great variations.

Surprisingly, the American structural practice, both as regards steelwork and concrete, seemed to be on a par with current practice in Britain 20 or 30 years ago. The only development was high-tensile bolts, which did not seem to be a world-shattering innovation. Apart from the question of fatigue, mentioned in the Paper, were there any economic or

material reasons for the Americans not developing the use of welded structures and prestressed concrete ?

In concluding, Mr Taylor Thompson said he felt a similar Paper on some Continental country would form an excellent foundation for discussion.

Mr Keith Brinsmead noted that in America they were anchoring long-welded rails only at the ends and, owing to the type of construction of the American track, using the dog spike, there was a relatively loose fixing between the rail and the sleeper elsewhere. In Britain there was a state of mild alarm with regard to the question of buckling, expansion gaps, etc. Did the Authors consider that the apparent immunity from those troubles in the United States was attributable to the deeper sleeper (7 inches against 5 inches) and possibly to the closer spacing of the sleepers ?

On the question of staggered joints, Mr Brinsmead's experience when travelling in the United States was that the even stagger did not produce any uneven running ; however that might be due to the superb springing. Experiments were in hand in Britain with a mild stagger, and on the Eastern Region there was a section of fast electric track where there was one section with the joints staggered two sleepers apart and the next section with the joints square. A recent report showed that there was every indication that this partial stagger was giving better riding and reduced wear and tear, as compared with square joints.

British railway engineers were gradually going to be forced away from complete renewals into partial renewals as more and more flat-bottomed track was laid, and it therefore became a matter of importance to study the American methods and see how those methods could be applied in Britain. The Paper was a valuable contribution towards such study.

Mr Brinsmead then showed a number of lantern slides of American machines which might be suitable for use on British Railways.

A typical re-railing scene was shown where one rail over a considerable length had been taken completely clear and the adjacent new rail was in position. For a short time there was lopsided running, with one new and one old rail.

Also illustrated was a fast light-weight crane used for handling rails.

Heavy rails could be positioned with a useful three-man machine (see *Fig. 24*, facing p. 308), which could easily be lifted out of the way. The gauge of the machine was approximately 3 feet, one rail being laid temporarily.

There was a power-jack for lifting the track and the operator had his level in front of him so that he could adjust the cant fairly quickly.

Further slides showed the McWilliams mole machine working in the 6-foot way, with its swing-out arm clearing the muck out of the way. There were no baskets such as were seen in Britain.

The Pennsylvania Railroad experienced considerable trouble from cinders. American locomotives being usually mechanically fired, they produced a tremendous amount of ash. After having tried a brush, the

Pennsylvania Railroad had finally produced what amounted to a vacuum cleaner, using steam.

Mr Campbell, when introducing the Paper, had mentioned American enthusiasm and zest, and Mr Brinsmead concluded his remarks by showing, as an expression of the American outlook on life, a picture of a weed-killer in action on the railroad.

Mr P. S. A. Berridge said that amongst the good ideas copied from abroad, he would include stepped bearings for medium-span girder-bridges and interchangeable unit steel trestling, which had been standard practice on Indian Railways 20 years ago, and the use of high-strength bolts instead of hot-driven rivets for field joints in structural steelwork and bridges.

High-strength bolts had been used in bridge work on the Western Region of British Railways for the past year. The bolts were of steel, having an ultimate strength of 45 to 55 tons per square inch with a minimum yield point of 34 tons per square inch. The nuts were of mild steel. Case-hardened washers, "65–70 Rockwell A," were placed under the head and the nut, and the bolts were tightened so that the steel in the shank was stressed not to the plastic range, but to within about 85 per cent of the yield point. When that had been done, the shear strength of the joint was developed by the grip of the bolts. There was no question of the shank making contact with the sides of the holes; the holes were in fact $\frac{1}{16}$ inch larger than the bolts. However, special attention was paid to the accuracy of the machining on the bearing surfaces of the bolt-heads and nuts, and those surfaces must be accurately square to the axis of the thread. Shanks were not machined. In a recent order for about 3,000 such bolts, the cost per $\frac{7}{8}$-inch-diameter bolt, $3\frac{1}{4}$ inches long, complete with mild-steel nut and two case-hardened washers, was 2*s*. 6*d*. That compared very favourably with hot-driven rivets which, in certain cases, cost anything up to 9*s*. a rivet. By using high-strength bolts, the need for field bolts was eliminated, though a few parallel drifts were necessary to locate parts before the bolts were tightened.

The bolts were tightened with ordinary spanners of sufficient length to enable the required torque to be applied by a man exerting a force of about 70 lb. The torque was checked for about 5 per cent of the bolts so tightened with a torque-measuring spanner fitted with a dial indicating the torque applied to turn the nut on the final tightening motion.

In bridgework there were many places where a spanner long enough to develop the required torque could not be used. To overcome that, torque-multiplying spanners giving a mechanical advantage of about 7 to 1 were under development (see *Fig. 25*, facing p. 308). But there again a complication arose because the torque-measuring spanner needed something against which the reaction could be taken. To overcome that, prolong bolts had been produced. The shank extended beyond the thread in the form of a spline, and the spanner fitted over the nut and the prolong. As the nut was turned, the reaction was taken against the prolong. That was an excellent

idea, for it ensured a truly axial load on the bolt; but there were certain snags. It was not so easy to use in tight corners. The prolong made the bolt longer, so that it needed more space to enter the holes. The spanner was necessarily deeper and more space was required to put the nut on and take it off. There was also the risk of damage to the prolong when the bolts were bagged. The spanner shown in *Fig. 25* incorporated a limiting device so that the bolts could not be over-tightened. When tightened to the required torque, the spanner came to a complete stop. It did not suddenly slip, and there was no risk of the operator hurting himself. A $\frac{7}{8}$-inch-diameter prolong bolt, $3\frac{1}{4}$ inches long, cost 9*d.* more than an ordinary high-strength bolt of the same size.

Other torque-multiplying spanners were being developed for use with ordinary high-strength bolts, and they would be tried and compared with the prolong bolt spanner to see whether it was worth while adopting prolong bolts.

Mr A. H. Toms said that in 1950 he had visited the United States as a member of the Economic Co-operation Administration Railway Technical Mission, and he was therefore most interested in the more recent impressions of the Authors.

The most significant difference between American and British rails was the higher carbon content and tensile strength of the former. Originally that had resulted in numerous transverse fissures in the rails, but they had now been virtually eliminated by the controlled cooling after rolling. A question which could therefore now be legitimately asked was—how would rails of British steel compare in performance with American rails when subjected to the much heavier American axle loads? Mr Toms had just been reminded that the steam locomotive was on its way out and that the Diesel locomotive did not produce such heavy axle loads. That was true, but it had smaller wheels and the stress concentration in the rail-head was augumented by reduction of wheel diameter.

Shelling of the running edge of American rails was a problem that was being investigated most ably by Cramer at Illinois University. It seemed to be a fatigue failure in shear comparable with the shelling of sorbitized rails used for a time on the Southern Region, but occurring in a different zone.

Mr Toms illustrated his remarks by means of lantern slides. *Fig. 26*, facing p. 308, showed 100-lb. sorbitized rails used at Earlswood. Failure had occurred by shelling down to the zone of maximum shear stress after irregular work hardening of the surface under traffic and had necessitated rejection of the rails when they were only slightly worn. Pieces shelled out to a depth of $\frac{1}{8}$ inch or a little more. That depth of $\frac{1}{8}$ inch was not fortuitous; it could be shown by calculation that it was the depth at which the maximum shear stresses occurred in the rail-head. Although, before collapse by shelling, the work-hardened areas appeared to consist of a series of irregular patches, if one looked obliquely along the rail the

normal type of regular pitched corrugations could be seen. *Fig. 27* showed where the breaking out by shelling on the top of the railhead was beginning to become continuous.

The effective life of a rail in service seemed therefore to be determined by fatigue endurance in local shear, and was not necessarily augmented by increasing the surface hardness. Was the end-hardening which was now being used widely in America going to result in shelling of that type? Although the American edge failure occurred in a different place, it was a manifestation of the same general phenomenon.

Incidentally, the way in which rail corrugations on the Pennsylvania Railroad were dealt with by the rail-grinding train, which removed them in one pass at 1 mile per hour, was very impressive.

Mr Toms referred to the drilling of bond holes in the side of the rail-head near the end. No adverse effects had been reported, and that seemed to indicate that American rail steel was tolerant of such stress raisers (admittedly not in the most highly stressed zone).

A piece of equipment which Mr Toms had handled in 1950 and was glad to have seen introduced into Great Britain was the Branson "Audigage." That supersonic apparatus seemed to have a big future for detecting rail-head flaws.

One could not fail to be impressed by the wide use of manganese-steel crossings in America, the care taken in making them, and their value in compact lay-outs.

Very good riding was obtained at exceptionally high speeds over track which in Great Britain would definitely be classed as "bad." Were the higher track standards maintained in Britain solely to compensate for deficiencies in rolling-stock design?

The success in America of large and small mechanical ballast-cleaners seemed to arise from the fact that most of the fouling materials were coal and other granular particles, which were easily sieved out. Where clay occurred in quantity, the plant was far less efficient and might need at least two passes with an interval for drying out.

Some of the items of mechanical plant which had been developed for complete mechanization of re-laying seemed unnecessary. A typical example was the machine for brushing the ballast off the top of sleepers prior to power adzing. The job could have been done with a hand broom just as quickly, and probably more efficiently. On the other hand, such items as the power-jacks for track lifting were excellent.

Some of the ballast-tamping machines were so violent in action that a considerable amount of pulverization of the ballast seemed to occur. That might not be entirely a disadvantage since shovel packing introduced smaller fragments, but those should be free from "fines."

After unsatisfactory trials of track grouting had been carried out on the Southern Region of British Railways, Mr Toms had consulted both Professor Peck of Illinois University, and Mr Rockwell Smith of the American

Association of Railroads' research team for any sound theoretical reasons for the beneficial effects of track grouting on many sites in America. They confessed to being unable to offer any, but when shown diagrams of the pre-grouting conditions of the cuttings at Clapham Junction and Haslemere where the British tests were made, they were not surprised at the lack of success achieved where so much confusion of the road-bed materials and choking with clay existed.

Mr Toms could not understand how the erratic seams of grout shown in *Fig. 8*, p. 282, could improve either the drainage, the imperviousness of the roadbed, or the strength of the clay.

It was understandable, however, that the forming, by means of explosives of sand-filled " camouflets " in the weak zones of waterlogged banks might facilitate drainage and increase the frictional resistance to further movement.

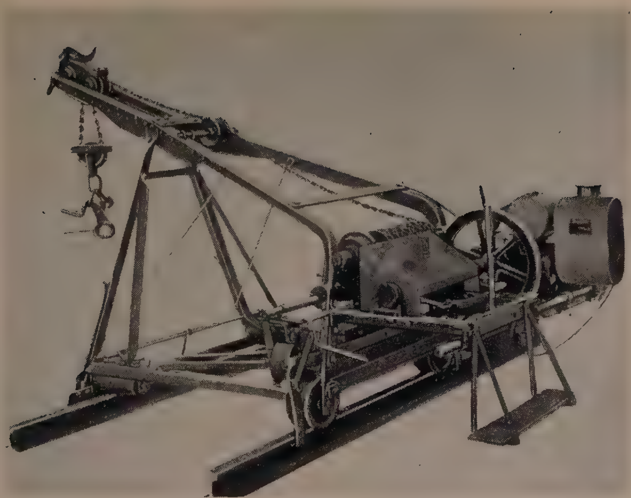
Mr Frank Turton said it was stated on p. 292 that wrought-iron floor plates were used on steel bridges, but on reading a little further he found that waterproofing the plates had obviously been quite an expensive process. What was the object in using wrought-iron in preference to steel ? Also, could the Authors give any idea of the relative cost of wrought-iron and steel plates in the United States ?

Mention was made in the Paper of the costly and difficult repair work which was now necessary on some existing concrete bridges constructed between 1920 and 1930. That appeared to be rather a coincidence, because in Britain trouble had occurred with reinforced-concrete bridges built during approximately the same period. From observations which had been made, bridges built before 1920 in reinforced concrete appeared to be still good. Mr Turton thought that the conclusion had been reached in Britain that the trouble was probably due to lack of cover on the main steel. Although the American Association of Railroads was carrying out an investigation into the deterioration of concrete, Mr. Turton wondered whether the Authors had been able to obtain any information relative to the causes of the failures which had taken place in America.

In Britain prestressed concrete was being used in many ways, some of them probably unwise ; and warning ought to be taken from the experience with reinforced concrete so that a similar position did not arise with prestressed-concrete structures within the next 20 or 30 years. Prestressed concrete should be used with reasonably sound engineering judgement and not on " fancy structures."

It was also mentioned on p. 292 that railway bridges were almost entirely of riveted construction, and that doubts were still expressed with regard to the fatigue strength of welded girders. There was a mass of information on that subject now, and Mr Turton considered that sufficient information was available to design with reasonable safety. A much bigger bogey now seemed to be brittle fracture in thick plates. Even so, whilst recognizing the existence of those problems, he wondered just how far the American

Fig. 24



The Three-Man Rail Layer

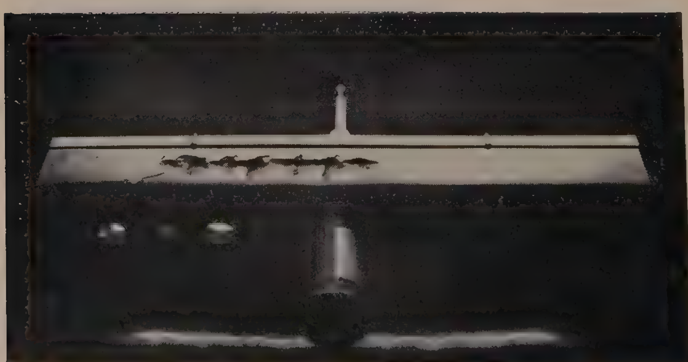
Fig. 25

Special Duty Spanner

Fig. 26

FAILURE OF SORBITIZED RAILS BY SHELLING

Fig. 27



SHELLING BECOMING CONTINUOUS

conservatism in that connexion was really born of economic considerations. Was there some vested interest in continuing riveted structures rather than having welded structures ?

The methods adopted for measuring strains in bridges in America were extremely interesting. Probably they were trying to fix a code of loading for the design of bridges in a similar way to the attempt made by the Bridge Stress Committee in Britain in the 1920's. There seemed to be no reason why British engineers should go to such elaborate lengths when there was the Bridge Stress Committee's code of loading which was still adequate for all requirements. Were the methods adopted for measuring the strains in bridges in America also intended for the assessment of the carrying capacity of existing bridges ?

Mr A. W. T. Daniel said that the differences between railway civil engineering practice in the United States and Britain usually arose from the different conditions prevailing. For instance, the American railways were very fortunate in usually having ample space for manœuvre so that standard track formations could be used almost everywhere. It was mentioned on p. 275 that on the few occasions when special trackwork was needed, it was designed and detailed by the contractor who fabricated the material. It would be interesting to know how those contracts were let, and on what basis payment was made, since the contractor would not know what he had to supply and lay until he had completed the design. Presumably there would be a lump sum for surveying the site and designing the lay-out, and a clause to the effect that the design must be approved by the resident engineer, who in his turn would have to consult the operating department.

The scheme by which university graduates joined the railway service as permanent-way apprentices, in due course being promoted to Permanent-Way Inspectors, thence to Assistant District Engineers, and so on, was very interesting, and was radically different from the methods in Britain. It had obvious advantages, but it seemed curious that design and construction of bridges and structures should be in almost another compartment, which offered less chance of promotion. There would appear also to be the additional disadvantage that ultimately the engineer, although having ample permanent-way experience, had little drawing office and design experience, which one would have thought was a handicap. The fact that transfers might be made to the Operating Department was very interesting, and in contrast to the practice in Britain.

With regard to the details of the track, it would be interesting to have the real answer to that perpetual puzzle, the 39-foot rail ; it seemed extraordinary that rails could be rolled up to 155 lb. per yard and yet never longer than 39 feet.

The question of welding long lengths of rail appeared to have been settled, and it was obvious that the expansion was prevented by compressive stresses in the rail ; but in view of British experience, even with

jointed track, it would be interesting to know what precautions, if any, were necessary to prevent buckling. Anchorages to prevent creep had been mentioned, and it would be interesting to know whether they assisted in the prevention of buckling, either sideways or in the vertical plane, whether extra anchorages had to be used for that purpose, or whether the natural stiffness of the flat-bottomed track was in itself sufficient.

Apparently, in spite of the use of flat-bottomed rail, the Americans had the usual trouble with battered rail ends and low joints. No mention was made in the Paper of the two-hole fishplate, which seemed to be going out of use everywhere ; in fact, six-hole fish-plates were used with sleepers nearly under the joint. Presumably the latter was the answer to rail-joint problems.

The Authors had stated that the baseplates were canted to 1 in 40 to take a worn 1-in-20 wheel, but the reason for that was not clear, since it would appear that the difference in the canting would accelerate wear.

On p. 280 it was stated that in the first motion of a vehicle over the rail the " hold-down " spikes were withdrawn, and that subsequently the only function of the spikes was to hold the rails to gauge, there being free vertical motion between the rail and the baseplate. The advantage of that was not clear for, under a load, the deflexion of the sleeper into the ballast could not be affected by the partial withdrawal of the spikes; the difference would occur on the upward movement of the rail on relief of the load. It would be interesting to know the magnitude of the upward deflexion of the rail relative to the baseplate, and whether it was ever enough to lift the rail clear of the shoulders of the baseplate, a situation which would be dangerous on curves, since the " hold-down " spikes would then be performing a function for which they had never been intended. It would seem that the movement of rail relative to baseplate was likely to produce battering and heavy wear, and it was not clear why that should be more desirable than movement of the sleeper in the ballast, since a necessary requisite of smooth running was an elastic road-bed. Pumping was usually serious only at the joints and that would be rectified by welding. The Authors had mentioned the use of anchors, the minimum number being eight per rail. That seemed to be very high, and it raised the question whether rail creep was not facilitated by the looseness of the rails in the baseplates. The question arose as to whether in claiming the loosening of the " hold-down " spikes as an advantage it was an attempt to justify what was really a weakness in the present design of flat-bottomed track and fittings.

It was also stated on p. 280 that the sleepers used in the United States gave 38 per cent more bearing area than in British practice, but since the sizes of the sleepers were the same, the reason would appear to be the much closer spacing adopted in the United States. It was an interesting speculation as to whether or not the success of American track arose from

the closer sleeper spacing, which covered up possible deficiencies in the flat-bottomed track.

Mr R. L. McIlmoyle said that in 1946 he had been privileged to see the American railways at work, and although he had been in America for only a few weeks he had managed to see quite a lot. He was now very interested in making a comparison between what he had seen then and what the Authors had seen five years later—in 1951.

Mr McIlmoyle said that he very much envied the Americans their American Railway Engineering Association. There was, unfortunately, no comparable organization in Britain. The Association's Manual was an invaluable item in a railway engineer's library.

The Authors had given some attention to the organization of the Pennsylvania Railroad but there was one point which they had not stressed sufficiently. Mr McIlmoyle understood that all district engineers, almost without exception, were given a period as district superintendents. At the end of that period they could either stay in the operating department or come back to the engineering department; but if they wished to proceed to a higher position in the engineering department, they must have first served on the operating side. That was an admirable idea. It made the man who came back to engineering a better engineer, and it made the man who became an operating man a better operator because he had an engineering background. Many of the difficulties in Britain might be removed if there was some interchange of that nature.

Mr McIlmoyle had had experience of staggered rail joints, and considered that the smooth riding in America was not due to staggered joints but to heavy American stock and the springing. It had already been mentioned that the speeds over the tracks would have been frightening in Britain, but the fact was that one went over them perfectly comfortably and did not realize that there was a bad track unless one was at the rear and saw what the train had just passed over! Mr McIlmoyle did not consider that staggered joints were the answer.

There had already been a reference in the discussion to concrete bridges. The experience of the Authors with regard to concrete bridges in America was not quite in accord with his own. Admittedly, a lot of very poor concrete had been placed in the period mentioned by the Authors, because at that time the placing of concrete by chute was very popular and it had gone down like porridge. One could not produce concrete that would last under those conditions; but many of the structures that had been placed before and after that period were in very fine condition.

Mr McIlmoyle thought that the largest concrete structure on the railways in America, Tunkhannock Viaduct, had been built in about 1914 or 1915. At the time of his visit, it had just been inspected very closely after being in use for 30 years without a penny having been spent on it, and he had been informed that there was no need to spend any more money on it because it was in perfectly good condition. He had seen a flat-slab

viaduct, about 3 miles long, which had been in use for 20 years carrying very heavy main-line suburban traffic. It was in perfectly good condition and, again, no money had been spent on it. Having seen quite a number of other structures and read many reports both on railway concrete structures and other types of concrete structures, he believed concrete had completely proved its worth in America.

Mr J. G. F. Inglis, referred to the section of the Paper dealing with freight-classification yards, and said that he could not agree that in most yards there were three stages of retardation. Two companies made the majority of the retarders in the United States. One of the two companies went in for three-stage retardation, and the other company, which made the electric retarder, went in for two-stage retardation. Having studied both of them, there seemed to be no case for three-stage retardation.

A contract for a new yard had just been let to a company working on two-stage retardation on a railroad which had previously taken its equipment entirely from a company working on three-stage retardation. The signalling equipment had been obtained from the same source. Tenders for the new yard had been invited and the company working on two-stage retardation had managed to get the contract, although the railroad had previously used the other company's equipment for its signals, etc., so that a considerable change-over in stores had been necessary. It was a tribute to the two-stage design that it had managed to win the day.

With regard to track maintenance, the Southern Railroad in the United States had just abolished throughout their entire system the traditional length-gangs. As an example of that, a permanent-way inspector in North Carolina was responsible for 140 track miles of permanent way formerly covered by one hundred men in gangs of six or seven up and down the line. Those gangs had travelled mainly on track trolleys, subject to traffic delays, and so on. The same length was now worked by two 12-man gangs, one nominally north and one south, both complete with their own trucks and trolleys and one 24-man extra gang. To put the scheme into operation, it had first been necessary to improve the access to the track by road. The railroad had made agreements with the farmers to get additional rights of access. The scheme was working well so far and had achieved a saving of 50 per cent in manpower.

It was the normal practice, as used in Britain on some branch lines, but the important difference was that the track inspection was now carried out by the permanent-way inspector and his assistant, and not by a patrolman. Normally the assistant rode over the track on a trolley, walking a little each day to cover the whole distance on foot. In that particular case he went north one day and south the next. The inspector himself usually rode on a train to do the inspection.

Mr Inglis had made several trips with the inspector and his assistant, who claimed that by using the system they could get better and more uniform inspection.

Another important point was that if a defect was discovered, the man who found it had authority to have it corrected at once, and did not depend on the report of his superior.

The Southern Railroad now had a policy of keeping all non-profit making traffic off the railroad by providing its bridge repair gangs and other extra gangs with trailer caravans which were towed around.

With regard to the stabilization of wet formation, the Southern Railroad believed in putting in sand, not by grouting, but by more direct means. They put in sand piles, as it were, 5 or 6 feet down, or perhaps more, and they reckoned that by getting about 20 per cent of the net surface area, they could deal with almost anything. They claimed that that was cheaper than grouting.

Mr Geoffrey Mottershead said that he had visited the United States in 1953 under the same scheme as the Authors.

The Authors had stated that it was in accordance with the principles of American track construction that free vertical movement on the base-plate seat was allowed, whilst the baseplate was fastened rigidly to the sleeper. Mr Mottershead did not believe it was common American practice to fasten the baseplate to the sleeper with a really rigid fastening. On one large American railroad the standard method was to have two cut spikes to hold the rail to line and to provide separate holding down spikes for the baseplate only on sharp curves and turnouts. That was by no means uncommon in America. There were some railroads which used separate screw spikes, but he thought they were in the minority. The consequence was that cutting into the sleeper by the baseplate was one of the most common causes of renewals of sleepers in the United States. Experiments had been carried out by the Association of American Railroads and it had been found that, by introducing two-screw spikes as separate hold-down fastenings, that cutting effect on the sleepers could be reduced by 30 per cent. However, it was still common practice simply to use the cut spikes which, after all, had never really been intended as hold-down fastenings in the first place.

The Authors had mentioned a new patent vapour-pressure method for seasoning sleepers. By that method one could season a red oak sleeper, which normally took 15 months, in 14 hours. In addition, one important aspect of that treatment was that the stresses which were normally associated with seasoning were very greatly reduced, and checking and splitting were thus reduced to a very marked extent. The process had now been in existence for several years, and eleven American railroads had installed test lengths of those sleepers and had found that splitting and checking were very considerably reduced.

A third cause of sleeper deterioration was splitting of the sleepers after they were installed in the track owing to weather conditions. One railroad had tried the method of spraying asphalt over the whole track, and it had been found that that diminished the splitting considerably; in

addition, it helped to prevent the creosote from leaching out of the sleepers and volatilizing.

On the subject of reinforced concrete, perhaps because he had been in America a year later than the Authors, it had seemed to Mr Mottershead that reinforced and mass concrete were enjoying a renaissance there. It seemed that modern methods had convinced engineers that durable concrete could be produced with a reasonable amount of supervision. He has seen quite a number of new structures being erected in reinforced concrete. An important point was that air-entrained concrete was now usually used, for bridges and structures as well as roads. The "Pre-pakt" intrusion method of concreting was being used in America for both repair work and new work. The piers of the Victoria Bridge over the St Lawrence River had been repaired by that method.

He could not altogether understand it when the Authors said that they had never experienced severe rolling on American track. On certain railroads there was severe rolling—but perhaps the Authors had not travelled on those railroads.

*** **Mr W. T. Everall** remarked that reference was made on p. 292 to a very important development in steelwork construction. It was the introduction of high-strength bolts, and especially their use in structures subjected to repeated dynamic loads and vibrations and where difficulty was experienced with ordinary power-driven rivets and bolts working loose.

Could the Authors say if it was the practice in the United States to paint the contact surfaces of the metal before tightening the high-tensile-steel bolts, or to leave such surfaces unpainted? From tests made in Britain it had been found that unpainted faying surfaces had approximately 40 per cent more resistance to slip between the plates after tightening the H.T.S. bolts to 85 per cent of their proof loading than those with painted surfaces. If a satisfactory paint had been found capable of resisting slip, what was its composition?

Mention was also made on p. 292 of the practical difficulty of applying the required bolt tensions. Mr Everall had been interested during the past 18 months in the development of a manually operated torque wrench. A geared wrench with a pre-set load of 550 lb.-feet had been successfully developed and was used for tightening the nuts of 1-inch-diameter high-tensile-steel friction bolts with unified fine threads. The wrench engaged with an extension on the bolt taken beyond the threaded portion. The extension acted as an anchor and permitted the nut to be tightened by the same wrench to the required torque. The tool obviated the necessity for using an auxiliary torque arm. The geared wrench could operate in confined spaces where a torque spanner, with its longer lever arm, could not be used.

*** This and the following contributions were submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

A further advantage of the tool was the impossibility of exceeding the pre-set load or of applying less than the set loading, because when the correct load was reached the operating handle was freed by a torque-limiting device embodied in the operating lever.

Mr G. D. S. Alley referred to p. 277 of the Paper and said that no mention was made of the top radius of the American rails. That was a very vital point. The 131-lb. American rail with its 14-inch radius was a good deal flatter than the British Railways standard 9-inch radius, and that might have some bearing on the statement made on p. 279 that the baseplates and fastenings were provided with the seat canted at 1 in 40.

Where relevant movement occurred between two materials, wear would take place. The Paper suggested (see p. 280) that if a sleeper remained solidly on the bed, wear would not take place between the baseplate and the top surface of the sleeper, no matter how hard and at what rate the rail was forced down on to the baseplate. Mr Alley considered that that American practice tended to increase the mechanical failures of sleepers.

Surely, to keep the track level, it was the rail which was required to be kept level, and if indentation took place on the top surface of a sleeper, it was more difficult to make good than the packing under the sleeper.

Reverting to paragraph (4) on p. 280, the third sentence appeared to contain some arithmetical errors, since 8-foot-6-inch sleepers were standard in Great Britain.

On p. 297, it was suggested that the practice of staggering rail joints appeared to have some merit. Various staggers had been tried out in Great Britain for many years. Half-length staggers to 60-foot rails were useful on very sharp curves. On the other hand, on main lines the resultant effect on passenger comfort was equivalent to the reintroduction of 30-foot rails, and was scarcely to be recommended. Again, to stagger rails by a few feet was found, at many speeds, to lead to more uncomfortable riding.

Mr Alley was astounded at the suggestion on p. 298 that the renewal of one mile of pre-assembled track required possession of two lines for 12 hours and speed restriction for 7 days.

The main reason for the introduction of a speed restriction was the disturbance caused to the ballast. The old London and North-Western system of carrying out plain line renewal on main lines on a weekday, covering $\frac{1}{4}$-mile per week with a re-laying gang, had much to be said for it. However, it did entail a speed restriction for longer lengths of line than when main-line renewal was carried out on a Sunday; four weeks' work with pre-assembly methods was then covered, occupation of the second line being required for only 6 hours, and the speed restriction being raised after 4 days, thus permitting better facilities to the Operating Department.

Mr Alley hoped that young engineers would not consider that track (now that rails had a longer life) should have its sleepers continually changed, thus wearing out the rail and possibly again leading to variation in gauge.

Mr A. M. Sims observed that gauge corner-shelling and failures of a

somewhat similar nature in the heads of rails appeared to be more common now than in former years. The Authors had referred to that, also to fatigue failures in the joint area, under the heading "Track Structure." Could the Authors give some information regarding the location of those head-shelling failures? Had they been found to occur, on the American railways, mainly on curves of about 1,000 feet radius and sharper? Any percentage figures for failures found on such sharp curves, on flatter curves, and on straights would be very valuable for comparison with other countries where failures of that kind had occurred.

In the Appendix to Mr Sims's Railway Paper No. 34[1] a method, based on Professor S. Timoshenko's theory, was given for calculating the torsion bending stresses in the rail-foot due to lateral wheel loads. In the Correspondence[2] on that Paper, Mr W. B. Dobie had made a very important contribution regarding the torsion bending stresses in the rail-head, which Mr Sims considered should be used by railway engineers as a guide, if the method were accepted (as it could be in theory) as applying to the railhead. Mr Sims showed in his reply that, with a net lateral load of 10 tons applied to a 90R rail-head by an XS class engine in India, the total stress, including torsion bending, lateral bending, and normal bending, but excluding eccentric vertical loading, would, in theory, reach 35·72 tons per square inch compression in the head of the rail. The range of stress would be from that figure to about 3½ tons per square inch tension and it was, therefore, not surprising that shelling failures occurred by fatigue when lateral forces of high magnitude were applied to rail-heads, as they were on sharp curves at speeds.

In Mr Sims's opinion, it was of the utmost importance to keep those lateral loads as low as possible. There seemed to have been little co-operation between permanent-way engineers and locomotive designers in that respect. He suggested that, just as co-operation had been largely achieved with regard to a limit for wheel hammer-blow loads imposed on rails, so should a compromise be arrived at for the net lateral wheel loads imposed on the rail-head by locomotive wheels when passing round a curve of given radius at a particular speed, with a known superelevation for the curve. The magnitude of the net lateral load had a greater effect, relatively, on the stresses in the rail than the vertical loading applied by the wheel tread. The design of the locomotive bogie side-control initial loading and the control-spring stiffness were largely responsible for the net lateral loads on the rail-head. Admittedly, the compromise was between wheel flange wear and the effect on the rail-head but, now that head failures by fatigue appeared to be more common than in the past, the case for co-operation in design did appear to be assuming great importance. In effect,

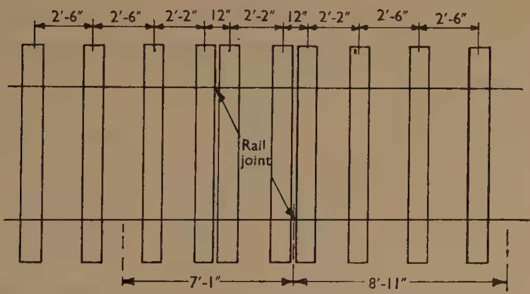
[1] A. M. Sims, "The Design and Strength of Standard Flat-footed Rail and Fishplate Sections." Railway Paper No. 34, Instn Civ. Engrs, 1949.

[2] Supplement to Engineering Division Papers for Session 1948–49. Instn Civ. Engrs, 1949.

the locomotive and the track could be regarded as parts of one " machine " or structural whole and they should not be designed as separate entities.

Referring to the Authors' suggestions for consideration under "(a) Track construction " (p. 297) regarding the staggering of rail joints by a few feet, Mr Sims's experience of that on the North Western Railway of India might be of interest. There, an experiment of that nature on straight track was carried out during the period from 1939 to 1941, but the results were not found to be satisfactory and the proposal was abandoned. *Fig. 28* gave approximate dimensions and illustrated the trial which had been made. Noticeable rolling had been observed and confirmed by the Hallade

Fig. 28



SLIGHTLY STAGGERED RAIL JOINTS

records. The reason for that was the difference in the support of the rail afforded by the sleepers on either side of the joint in each rail. It would be seen that, on one side of the joint, the support afforded by four sleepers was spread over a distance of 7 feet 1 inch but, on the other side of the joint, similar support was spread over the greater distance of 8 feet 11 inches. That inequality, being staggered for each rail, set up undesirable rolling. It was true that, in curves on the North Western Railway, joints were staggered approximately at the centres of full rail-lengths, using closers not less than 12 feet long on the inner rail, but that was the standard practice in order to maintain good alignment on curves.

Mr Campbell, in reply, dealt first with three topics which had been mentioned by several contributors to the discussion, namely, long welded rails, partial renewal of track, and the American practice of allowing free movement between rail and baseplate.

The main doubts about the use of welded rail in Britain seemed to centre round the problem of distortion of the track. That problem had, apparently, been overcome in the United States in spite of wider temperature-ranges. American practice was to use normal track fastenings, and

to anchor the welded length very heavily at each end and fairly heavily throughout its length. An analysis had been made of the forces produced in a given length of rail by normal temperature variations. Experiments had shown that, for a 22-inch spacing of sleepers in stone ballast, the resistance of the sleepers to longitudinal movement was 500 to 800 lb. per sleeper. It was thus possible to deduce the number of sleepers which should be anchored at the end of a welded length to prevent movement. Intermediate anchorage was determined empirically to prevent creep and to guard against the opening of a wide gap in the case of rail failure at low temperatures.

The present A.R.E.A. recommendation for anchorage on track carrying traffic in one direction only, for long welded lengths (that was, longer than 1 mile) was: anchors on both sides of every sleeper for about 250 feet from both ends, intermediate anchors on every other sleeper in the direction of traffic and a "back-up" anchor on every fourth sleeper. It was emphasized that the cut-spike fastening offered no resistance to longitudinal creep or temperature variation; anchors had to provide all the restraint that was required.

It was not the usual practice to increase the standard ballast shoulder of 6 inches, although a few companies did increase it up to 12 inches. It was recommended that rails be anchored at a rail temperature of 70°–90° F., although laying had been carried out at various rail temperatures between 28° and 130° F. Major maintenance works, including skeletonizing of the track, were carried out at temperatures lower than the anchorage temperature, otherwise only minor repairs were allowed.

Figures recently published by the A.R.E.A. showed that a saving in maintenance of about 30 per cent was being obtained by some companies, but it would be unwise, with the limited data so far available, to rely on that figure. There was, however, no doubt that a considerable saving could be obtained.

A number of speakers had mentioned the different rates of wear of rails and sleepers, which would make a policy of partial renewal appear to be more economical than one of indiscriminate complete renewal. It should, however, be mentioned that the closer spacing of sleepers in American track meant that one rotten sleeper would have a less damaging effect than would be the case in British track.

Mr Alley had expressed astonishment at the figures quoted for restrictions and possessions required for one mile of pre-assembled re-laying. A study of the various jobs quoted in Report No. 2 of the Relaying and Costs Committee, 1952, showed that the figures were conservative. The normal procedure, where no ballast had to be loaded up, was for the restriction to be imposed on the Thursday morning before re-laying when the length was opened out, bolts run, etc. Possession was taken at 6 a.m. on Sunday and retained until 5–6 p.m. Full speed was restored on the following Wednesday evening after tamping had been completed.

In support of partial re-laying Mr Campbell presented the following theoretical analysis of the two methods of re-laying one mile of track.

Prefabricated Re-laying

Two-crane renewal. (Based on the 1952 Report of the Relaying Methods and Costs Committee. Jobs 105, 106, 114, and 115 equated to a mile.)

Work at depot	1,136	man-hours
Work at site (excluding ballasting)	527	,,
Ballasting: with possession	441	,,
with restriction	3,332	,,
Total	5,436	,,

That excluded travelling time, lost time, inspectors' and look-out men's time.

This total might be expressed in another way :—

Off-site work not affecting traffic :	1,136	man-hours	(21%)
Site work under possession :	968	,,	(18%)
Site work with restriction :	3,332	,,	(62%)

Partial Renewal

(1) Re-railing and plating.

Assuming that one mile could be laid by 80 men in 7 working hours, including laying out new and picking up old material, the total time amounted to 560 man-hours.

(2) Re-sleepering at 6 sleepers per man day

$350 \times 8 = 2{,}800$ man-hours

Allowing 25% for laying out and picking up material

Total $=$ 3,500 man-hours

That could be expressed as :—

Site work with possession :	560	man-hours	(14%)
Site work. No interference to traffic :	3,500	,,	(86%)
Total	4,060	,,	

The difference in man-hours might be accounted for by the betterment achieved by forking out ballast and by getting a small lift on new ballast, but it appeared from the above analysis that partial renewal was no less economical than complete renewal and that there was a very considerable reduction in the interference to traffic.

Various criticisms had been made of the American practice of allowing free movement between rail and baseplate. The spike fastenings used allowed free movement of the rail but also movement of the baseplate on

the sleeper. That undoubtedly led to heavy mechanical wear on the sleeper. Many sleepers seen were very heavily plate cut until finally they broke under the plate seat as a result of frequent adzing to provide a level seat; that might be permissible for a 7-inch sleeper but not for a 5-inch sleeper. The present trend of research was to continue to use the cut spike as line spike and a more positive hold-down fastening such as a screw or a spring spike. Alternatively, efforts were made to reduce mechanical wear by the introduction of pads.

No difficulty was experienced with the rail lifting out of its seat even on curves, although a slightly deeper lip of $\frac{7}{16}$ inch was provided.

Pumping of the sleeper was not eliminated but was considerably reduced, since it was no longer lifted out of its bed in front of a wheel, allowing water and mud to be sucked into the void.

It would appear that the practice of allowing movement of the sleeper in its bed, which necessitated a tremendous amount of tamping and measured shovel packing, might be less economical than allowing free movement of the rail with a possible increase of rail gall and of mechanical wear of the sleeper.

Mr Cantrell had mentioned the testing of rail joints. The test cars were used for testing the rails between the joints and the hand supersonic instruments had been designed for joint testing only, although they were being marketed in Great Britain as general rail-testing instruments. They were good instruments and obviated the necessity for removing fishplates annually for the examination of rail ends. There were various opinions about joint lubrication in the United States, but Mr Campbell was not aware of any company which removed the fishplates for examination or lubrication; he was of the opinion that sound anchorage of the track made expansion, creep, and distortion of the track far less of a problem than it was in Britain, although sleeper depth and spacing undoubtedly helped, as suggested by Mr Brinsmead.

Various tests had been made of reinforced-concrete sleepers, but none, as far as was known, of prestressed sleepers. There was not the same economic inducement to find an alternative to timber sleepers when timber was so readily available in America.

Mr Cantrell had pointed out the differences in the sections shown in *Figs 8*, p. 282, but the two sections had been taken at different places on the track as was shown on the plan.

A number of speakers had mentioned the riding qualities of trains on track with staggered joints. It was not suggested that good riding was entirely attributable to staggered joints; that might be only a contributory factor. However, it did not appear to produce bad running in the form of a rolling action. Mr Brinsmead and Mr Sims had reported two trials of joint staggering, one successful, the other unsuccessful, which would suggest that no firm conclusions could be reached.

Mr Sims had asked for details of rail failures related to track curvature.

So far as Mr Campbell knew they were not available ; A.R.E.A. statistics were related only to the type of failure, type of rail, etc. Most railroads related failures to the cumulative tonnage of traffic passing over the rail. On one major railroad most rail failures occurred on a stretch of 40 miles with a high concentration of traffic but no severe curvature. However, lateral forces would undoubtedly contribute to the local shear failure mentioned by Mr Toms. The Americans did experience a form of shelling in a longitudinal direction at rail ends and that did develop into a shelly failure unless rails were "slotted" (cross-ground to a bevel). However, that should not condemn rail-end hardening, for normal rail-end batter was frequently the governing factor in rail renewal.

Mr Daniel had asked about contracting for special track work. The normal procedure was for the railway company to survey the site and provide a single line-drawing showing the crossing angles, etc., accurately. The contractor would then manufacture the units to his own design, approved by the railroad. The trackwork would normally be installed by the railroad.

The only explanation available for the use of a 39-foot rail was that the standard wagon bodies were 40 feet long and the rails had to go inside them. 78-foot rails could be rolled and were now being used in comparatively small quantities.

Mr Daniel was a little optimistic in suggesting that all joint troubles would disappear with the use of a 6-holed fishplate and a supported joint. A more probable solution would be to eliminate the joint by welding. The two-holed fishplate had never been used in the United States and had been abandoned in Britain for some time.

Mr Campbell agreed with Mr Inglis that two-stage retardation in freight yards appeared adequate. Most of the yards which he had seen had three-stage retardation and it was apparently considered essential, although the primary retarder was used only for cuts of several heavily loaded wagons.

The abolition of length gangs had been achieved by several American railroads but it was doubtful whether that would be advisable in Britain where access to the track was difficult and where local duties, such as point oiling, fogging, snow clearing, etc., were frequently necessary.

Two installations of sand piles had been made in Britain, one north of Aylesbury on the old Great Central line and one south of Berwick on the East Coast Main Line. One pattern of piling used on the former installation was apparently still effective but other parts of the length were deteriorating.

Mr Mottershead had mentioned the reduction of splitting achieved by the vapour drying process of timber seasoning. It should, however, be pointed out that the standard of a sleeper imported from Canada and the United States to Britain was very much higher than would be required by an American railroad. Many new sleepers were being put into American

track in a condition which would cause them to be taken out of British track as scrap.

Mr Alley had drawn attention to an apparent arithmetical error on p. 280. The intended meaning of the sentence referred to was that twenty-four 9-inch-by-7-inch-by-8-foot-6-inch sleepers to a 39-foot rail gave a 38 per cent greater bearing area per yard of track than twenty-four 10-inch-by-5-inch-by-8-foot-6-inch sleepers to a 60-foot rail.

Mr Nicholls, in reply, said that the most important point with regard to the use of materials had been expressed in the A.A.R.'s estimate that of all the material used in bridging, 40 per cent was timber, 40 per cent steel, and 20 per cent concrete and masonry. That was particularly true of the western railroads, where the timber was readily available and was certainly the cheapest and most economical form of construction.

There had been considerable criticism in the discussion of the Authors' remarks on reinforced concrete. He could give an assurance that most of the bridges he had seen on one particular railroad had been in a very bad state. He would admit that in the section of the railroad with which he had been concerned they had been subject to severe frost conditions—down to 40 degrees below zero. The Portland Cement Association, which had a separate department for railway work, had been carrying out tests with air-entrained concrete; he had seen some of the samples which had been under test for some time, and undoubtedly there had been a great improvement in the resistance to frost action of the air-entrained concrete. Unfortunately, there was a slight reduction in strength, but he did not think that was serious compared with the advantage gained from the frost resistance.

Another problem in connexion with the reinforced and mass concrete was the spalling of the concrete at construction joints. He had seen that on a number of structures, particularly on the abutments of bridges. Where pouring had ceased for a time, there was always a joint which was never made good, and the concrete spalled at that place.

It was interesting to note that, in a recent test carried out in the laboratory of the United States Bureau of Reclamation at Denver, on a deteriorated bridge slab removed after 40 years' service, the ultimate load was $3\frac{1}{2}$ times the design load. If a satisfactory method of applying a protective concrete cover to the steel reinforcement under traffic could be devised it would appear that the life of such structures could be considerably increased, although the cost might be high. The intrusion pre-packed concrete mentioned by Mr Mottershead seemed to have very great possibilities for under-water placing of concrete. The aggregate was placed within the shutter and the grout was pumped from the base of the shutter displacing the water without mixing with it.

He had been very pleased to hear from Mr Berridge about the use of high-strength bolts. The Committee on riveted and bolted joints in the United States, which included representatives from all steel firms,

universities and the railways, had been carrying out a very extensive programme of tests on riveted and bolted joints. It had been shown conclusively that the high-tensile bolts gave a far greater resistance to fatigue than the hot-driven rivets. That had also been borne out by several installations in particular bridges. In the test specimens seen by the Authors, the contact surfaces of the plates were not painted and it was understood that that was the general practice.

The bridge-testing apparatus of the A.A.R. had been used both for determining a design code and for assessing individual bridges. The same apparatus had also been used for determining the stresses in rail arising from the passage of loads, and it was on the result of those tests that certain modifications had been made in the rail sections standardized by the A.R.E.A. The radii at the top and bottom of the web had been increased, for it had been found that the stresses were exceptionally high there. The rail sections given in *Fig. 1*, p. 276, showed that the British standard rail section had rather a sharp radius of curvature at those points.

With regard to the question of the use of welded girders; it seemed that it was the conservative attitude of the railways, and not any vested interest, which had resulted in all new work still being in riveted designs. At the University of Illinois, which was particularly interested in the question of fatigue, there were a number of machines of different types, and the latest type was one in which the machine was adapted for varying the intensity of loading. The aim was to produce the effect of cumulative fatigue which occurred under loading, say, on a cross-girder in a bridge where there were a number of heavy axles passing followed by a number of lighter-loaded axles.

The reason that prestressed concrete had not been used was that the cost of the traditional materials was less than that of prestressed concrete.

He had been very pleased to hear Mr McIlmoyle refer to the A.R.E.A. Manual. It contained information on practically every branch of engineering connected with railways. He felt sure it would pay many railway engineers in Britain to become members of the A.R.E.A. if only to obtain a copy of the Manual!

He had been interested to hear Mr McIlmoyle's remarks about the interchange of engineering and operating staff. That undoubtedly produced far greater co-operation between the two departments. If a district superintendent had been an engineer, he would appreciate, far more readily than a man who had not been concerned with the engineering work, many of the engineering difficulties, especially with regard to occupations.

Mr Inglis had referred to retarder yards. Mr Nicholls had himself been interested in the Milwaukee's Airline yard, which had 25 classification tracks and two-stage retardation, with automatic speed control in the

first retarder; but one operator had to operate all the controls. Mr Nicholls had noticed that in the Milwaukee's latest yard, at Bensenville, which had 70 classification tracks and three-stage retardation, the operation had been split up, so that one man operated all the switches, and two men operated the retarders.

ROAD ENGINEERING DIVISION MEETING

2 March, 1954

Lieutenant-Colonel G. T. Bennett, Member, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 44

"British Highway Bridge Loading" *

by

William Henderson, B.Sc., A.M.I.C.E.

SYNOPSIS

The Paper opens with a brief history of live-loads specification for the design of highway bridges in Britain. Between the wars, two design loadings existed. These were the British Standard Train and the Ministry of Transport Loading Curve.

Changes in the numbers and weight of vehicles have made it necessary to reconsider the live load to be used in bridge design; new regulations are being considered by the British Standards Institution and the Ministry of Transport, in collaboration, to meet these changes; the basis of these is discussed in the Paper. Highway bridge loading, it is suggested, should consist of two parts: the first, a design loading for bridges intended to carry normal traffic; the second, for bridges intended to carry traffic which includes heavy abnormal indivisible loads. A uniformly distributed lane loading, together with a knife-edge load, based on the former Ministry of Transport Standard Loading Curve is proposed for the former purpose, and a train of four axles, each axle having four wheels, for the latter.

HISTORICAL

UNTIL the nineteenth century, live loads for the design of bridges were not a problem to be considered by the bridge builder. The traffic carried by bridges was light and unimportant in comparison with the dead load. In any case, a sufficient knowledge of the theory of structures was not available, so that design loads would have been of no value.

In the latter half of the nineteenth century, however, the development of the traction engine had brought with it the need to build highway bridges able to carry quite considerable loads. Various recommendations were made; for instance, Professor Fleeming Jenkins, about 1875, suggested a load of one cwt per square foot, together with a wheel loading amounting to "perhaps ten tons on each wheel on one line across the bridge."

In the early twentieth century, Professor Unwin suggested a load of 120 lb. per square foot, or the weight of a heavily loaded waggon, say, 10 to 25 tons on four wheels. He later suggested a distributed load of some 80 to 120 lb. per square foot, or, in manufacturing districts particularly, a four wheeled waggon weighing some 30 tons.

The 1914–18 war marked the beginning of the new highway era. Demands made by the Armed Forces for mechanical transport provided the incentive for rapid development. The Ministry of Transport was brought into being immediately after the war and in 1922 introduced its Standard Loading Train (*Fig. 1*), which included a flat-rate allowance of 50 per cent for impact.

In 1932, a new approach to the design loading for highway bridges was introduced.[1] This was the well-known Ministry of Transport Standard Loading Curve. The advantages to the designer in using the loading curve were obvious ; based on the standard train plus impact, it greatly simplified the application of live loading in design. In view of the improvement in the springing of vehicles, the total impact allowance was considered to diminish as the loaded length increased, whilst a reduction in intensity of loading with increasing span was also recognized.

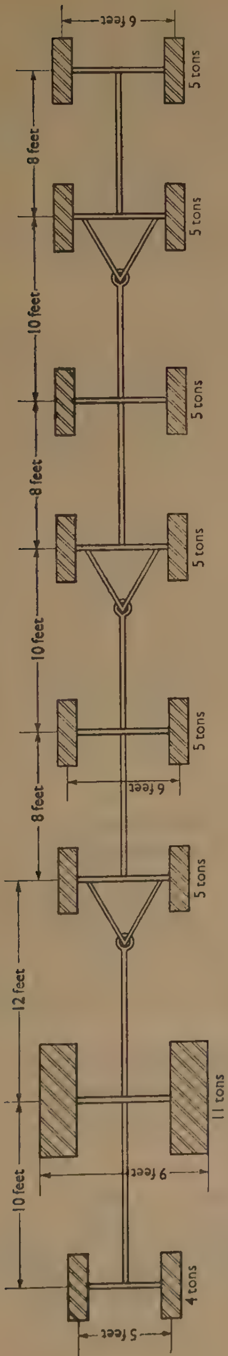
Contemporary with the Ministry of Transport Standard Loading Train and the Loading Curve there was the British Standard Train of B.S. 153 (*Fig. 2*). This train was similar in most respects to the former, fifteen units being specified for conditions similar to those applying in Great Britain. An impact factor, diminishing with increases in span, was specified with this loading.

General : Modern Traffic

In recent years, a considerable change in the weight and nature of traffic using bridges has been evident. The changes have been of two sorts ; the first is a general increase in the weight and number of normal industrial vehicles in use. These can conveniently be referred to as "legal" vehicles, and are governed by the regulations of the Statutory Instrument, 1951, No. 2101.[2] For the present purpose, these can be defined broadly as ordinary lorries up to a maximum weight of 22 tons. The second change has been the increase in number and weight of vehicles carrying abnormal indivisible loads. These are governed by the Statutory Instrument, 1952, No. 2173.[3] At first sight, this Statutory Instrument limits the weight of such vehicles to a maximum of 150 tons. It is possible, however, for hauliers to obtain a special order to move greater loads.

Even for comparatively light loads, that is to say, of the order of 30 to 40 tons, as well as for the much heavier loads, the type of vehicle used conforms to a broad standard. These abnormal load-carrying vehicles are generally well-deck trailers, having one axle at front and rear for the lighter

[1] The references are given on p. 350.



Loads shown are actual wheel loads plus 50 per cent impact

Fig. 1

Ministry of Transport Standard Loading Train

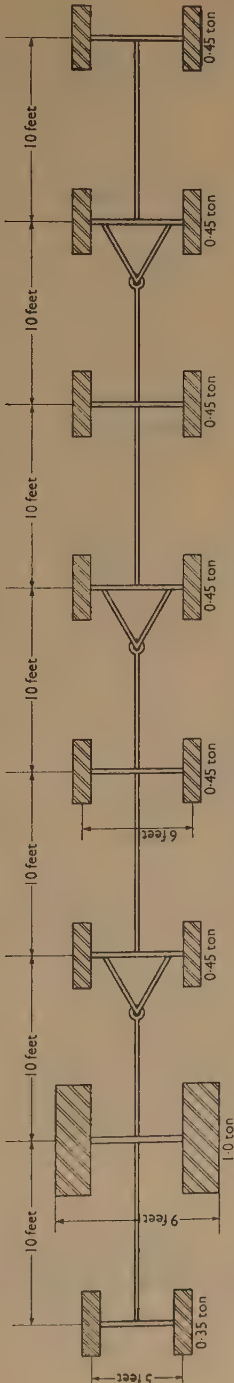


Fig. 2

British Standard Unit Loading Train

Impact to be added

weights, and a two-axle bogey at each end for the heavier loads. Recent modern designs, of which there are about three examples in existence, have pneumatic tires and two or three axles at front and rear ; these are intended to carry exceptionally heavy loads. In almost every case, and certainly always where heavy loads are concerned, these axles have four wheels each, the overall length of the axle lying between 7 feet 6 inches and 10 feet. In adopting pneumatic tires, the tendency is to use a wider axle, and the 10-foot length is likely to be fairly characteristic in the future.

The position is, therefore, that today both ordinary traffic and abnormal vehicles are dissimilar in weight and arrangement of wheels to those represented by the former loading trains.

Of the abnormal load carrying vehicles, the group weighing, say, 150 tons and more is not likely ever to become numerous, and the journeys they can be expected to make will generally be confined to a limited number of roads, so that they can be treated as a special case ; where necessary, bridges can be specially strengthened or designed for them, and, in doing so, it can reasonably be expected that when these vehicles are crossing a bridge, precautions will be taken to prevent heavy normal traffic being on the bridge at the same time.

There are, however, a great number and variety of less heavy, but still technically " abnormal," vehicles for which it is not possible to say that their journeys can be restricted to a limited number of routes. These vehicles, with laden weights ranging from the maximum 22 tons of the " normal " class up to about 80 to 100 tons can be expected to make demands on any of the main roads. For the purpose of highway design loading, they require to be taken into consideration in the design of main-road bridges.

At the present time, the Ministry of Transport and the British Standards Institution committee which is revising B.S. 153 (Steel Girder Bridges) are reconsidering the specification of live loads for the design of highway bridges. It is their aim that both bodies should adopt similar regulations ; their problem is, therefore, twofold. First, to produce design loadings which are simple to use but which also reflect, without wasteful excesses, the effects of actual traffic ; and secondly, to make these regulations sufficiently flexible so that they can be adapted readily to varying conditions at home and abroad.

The remainder of the Paper is devoted to a general statement of the approach which is being adopted to deal with the various aspects of the problem.

Normal Traffic and Abnormally Heavy Vehicles

Vehicular traffic is readily sub-divided into two classes—normal and abnormally heavy. So far as bridges are concerned, their effects, particularly on medium spans, are dissimilar. Normal " legal " traffic will,

in general, have its worst effect on the structure when the whole carriageway is congested with vehicles. These vehicles, as a result of international agreement, are, in most countries, limited to approximately the same upper limits of gross, axle, and wheel weights, namely, 22 tons, 8 tons, and 4 tons. Any variation between one country and another will be, essentially, one of traffic density.

Abnormally heavy vehicles, on the other hand, are sufficiently rare to make it unnecessary to consider them more than one at a time ; in fact, it is illegal for two such vehicles to pass on a bridge. Their worst effects will be those of two solitary concentrations. The maximum weight will differ from country to country and from route to route, but the general arrangement of the load will not differ widely.

It seems best, therefore, to consider the two kinds of loading separately, and to make separate specifications for them. All bridges will be required to carry normal traffic, but this cannot be said of abnormal traffic. Designs should, therefore, be made on the basis of normal loading, and thereafter be checked for abnormal traffic of the weight required.

Selection of Type of Design-Loading for Normal Traffic

A train of vehicles or an equivalent distributed load is the obvious selection for this purpose. Trains have the disadvantage of being awkward and cumbersome to use ; when they are specified it is common practice to compute equivalent uniformly distributed loads as a substitute. If, therefore, a simple distributed load can be developed to cover all conditions without being uneconomical, it is clearly preferable. The ready favour given in the past to the Ministry of Transport Loading Curve with its knife-edge load confirms this assumption and indicates a suitable form.

Effect of Number of Carriageway Lanes

The density of traffic on a wide bridge will be less than on a narrow one. Experience indicates the extreme improbability of more than two carriageway lanes being filled with the heaviest type of traffic. Whilst it is impossible to quote figures giving a firm quantitative basis for the proportion of load which will be carried by the remaining lanes, it is likely to be less than half that which is imposed on the most heavily loaded.

If, therefore, it is assumed that any two lanes on a bridge are fully loaded, whilst the remaining lanes carry half this loading, the total effect o na bridge should not be underestimated.

Effect of Width of Carriageway Lanes.

Carriageway lanes are now generally made 11 feet wide. On occasion, however, this width may be increased to 12 feet, particularly on curves ;

there is also the possibility that these lanes may be as narrow as 10 feet on bridges carrying roads likely to be used by heavy normal traffic. It is considered that ordinary traffic will, on busy roads, arrange itself in columns, such that there is one of these per lane of from 10 feet to 12 feet width. For instance, it is extremely unlikely that, in a width of 24 feet, three 22-ton lorries, themselves about 7 feet 6 inches wide overall, would travel or stand abreast, and even more unlikely that they should be in line in this position, with their heavy axles simultaneously over the worst position they could occupy for any one member. The probability that this would be repeated in succession along the bridge is negligible. It therefore seems reasonable to assume that only one vehicle will occupy the width of any normal lane.

On the other hand, where lanes are less than 10 feet wide, it is possible that traffic might be forced into occupying this narrower space, but if so, the vehicles involved are likely to be somewhat lighter than those which might occupy normal lanes. In these fairly rare cases, some reduction in design weight per lane seems appropriate.

Taking these points into consideration, a suitable solution for design purposes seems to be to provide a lane-loading which is unaffected by the width of the lane, provided this lies between 10 feet and 12 feet; where lanes are less than 10 feet wide, the lane loading could be reduced *pro rata*, on the basis of width of lane divided by 10 feet.

Distribution of Normal Traffic Within the Width of a Lane

On railway bridges, the exact position of application of the live load is known. Highway vehicles, however, may occupy any position in the width of a bridge. The following points are relevant to longitudinal beams, stringers, and cross-beams (reinforced-concrete slabs are considered elsewhere) :—

(1) On short-span members it is possible that the traffic in two adjacent lanes may lie close together, so bringing a disproportionate amount of load on one beam. It would be safe to assume, however, that the wheels of adjacent lorries would not be nearer to each other than 3-foot centres.

(2) This condition would not be likely to repeat itself for any considerable distance ; it seems a safe assumption that the upper limit for this length would be less than 75 feet.

(3) For lengths greater than this, it is reasonable to assume that vehicles in one lane will be in random positions within its width, and that they will not coincide with vehicles in neighbouring lanes in longitudinal position.

On short spans it will, therefore, be necessary to take into consideration the effect of parallel vehicles lying close to each other, with their near

wheels, say, 3 feet apart. As a consequence of this, the spacing of the beams will also require to be considered. Beyond 75 feet span, it seems reasonable to ignore this condition and to make a direct comparison between total effects for lanes of design loading and lanes of vehicles.

For longer spans, the foregoing considerations justify an assumption that an equivalent distributed lane-loading can be spread over the full width of the lane. The spacing of members in longer spans will generally be at least the width of a lane—a condition which provides further justification.

Due allowance must, however, be made for the width of lanes in considering shorter-span members, where the spacing cannot be ignored. The assumption has the merit that it will not penalize the use of wider lanes as would be the consequence of a design loading specified on a square-foot basis. It also makes some allowance for the fact that traffic on wide lanes will probably be travelling rather faster than would be the case on narrow lanes, and will consequently be more widely dispersed.

Longitudinal Distribution of Lane-Loading for Ordinary Traffic.

Any attempt to state a sequence of vehicles representing the worst concentrations of ordinary traffic which can be expected must be a guess. In the absence of data, the arrangements of vehicles shown in *Figs 3* have been taken as giving, for various spans, a representation of particularly severe conditions of loading in one lane.

Up to 75-foot length, 22-ton lorries with a very short wheel base, at the closest possible spacing have been assumed.

Taking into consideration earlier conclusions, namely :

(1) up to 75 feet, traffic in adjoining lanes should be considered to be 3 feet apart, centre to centre of wheels ;

(2) beam spacings should be taken into consideration over this range ;

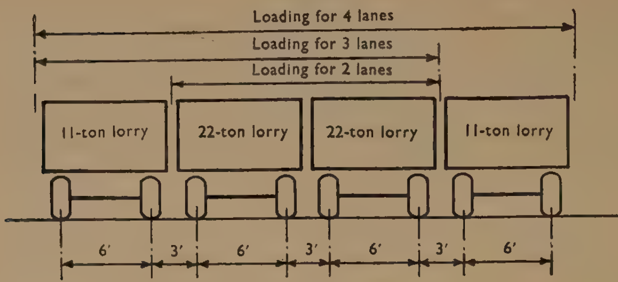
(3) the total design load per linear foot of lane should be unvaried for lane widths between 10 and 12 feet ;

a comparison has been made between values taken from the former Ministry of Transport Standard Loading Curve from 20-foot to 75-foot span and two trains of 22-ton vehicles in adjoining lanes.

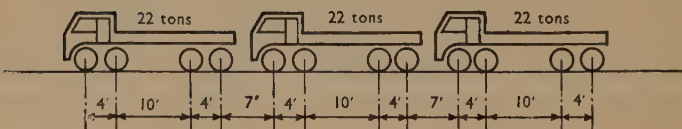
Table 1 shows the ratio :—effects of vehicles ÷ effects of design loading, for moment and shear, assuming that the lane loading for design is that for a 10-foot width on the former super-foot basis, but that it has been spread over the maximum lane-width of 12 feet. In arriving at these values, wheels have been treated as point loads, and no redistribution, or dispersal of load has been taken into account.

A minimum width of design load carried by a beam, equal to half the

Figs. 3



MAXIMUM TRANSVERSE DISTRIBUTION OF NORMAL TRAFFIC. ASSUMED ARRANGEMENT TAKEN FOR MOMENT IN CROSS-GIRDERS



LONGITUDINAL ARRANGEMENT FOR SPANS UP TO 75′-0″

LONGITUDINAL ARRANGEMENT FOR SPANS 75′ TO 500′

Five vehicles, each 22 tons, occupying 40 feet followed and preceded by four vehicles, each 10 tons, occupying 35 feet and further vehicles, each 5 tons, occupying 35 feet to fill span

VEHICLE ARRANGEMENT CONSIDERED FOR NORMAL LOADING

tane width has been assumed. This is similar to the former restriction in the Ministry of Transport Loading Curve, which imposed a minimum of 5 feet. The restriction is necessary because, below this spacing, the amount of wheel loading does not diminish as rapidly as would the proportion of distributed load carried by a beam.

The Table demonstrates that throughout the range, the effects of the former Ministry of Transport loading, converted into lane loadings, lie very close to those of the assumed vehicular traffic.

In addition to the values given for various spacings, Table 1 also gives the ratio for the total effects of a single lane of vehicles compared with those for a single lane of design loading. The amount by which this falls short of corresponding values elsewhere in the Table effectively demonstrates the need to take the spacing into consideration in the shorter-span members.

TABLE 1.—LONGITUDINAL BEAMS OF 20 TO 75 FEET SPAN

$$\text{Ratio}: \frac{\text{effects of train of normal vehicles}}{\text{effects of proposed design loading}}$$

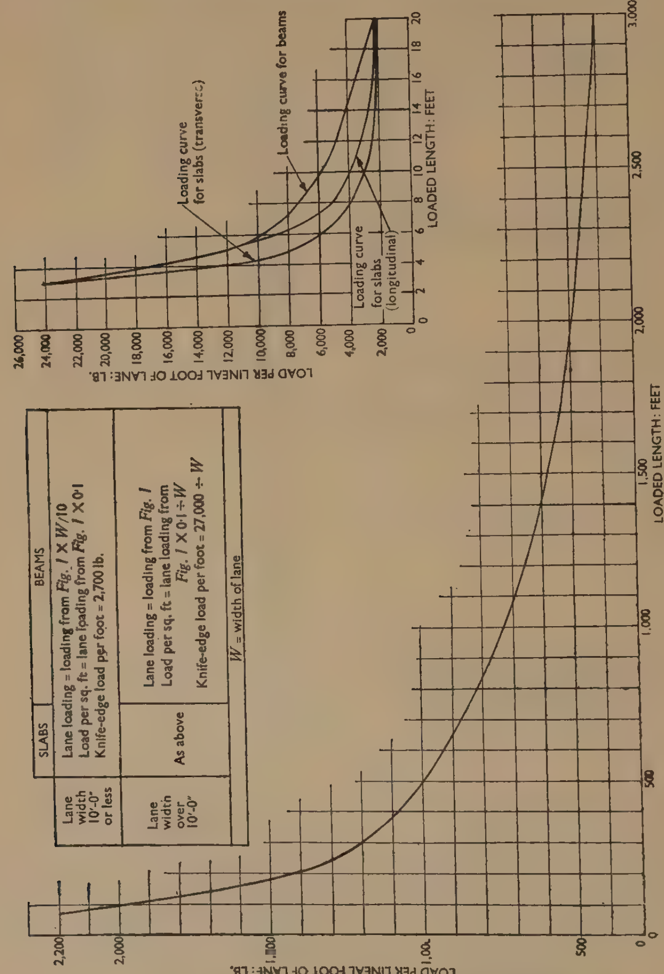
Span: feet	Spacing of beams and lane width	Ratio for moment	Ratio for shear
20	5 feet under 10-foot lanes, that is, ½-lane width	0·87	1·05
30		0·86	1·04
40		0·84	1·09
50		0·84	1·09
60		0·92	1·12
75		0·98	1·11
20	6 feet under 12-foot lanes, that is, ½-lane width	0·94	1·13
30		0·93	1·13
40		0·90	1·17
50		0·90	1·17
60		0·99	1·20
75		1·04	1·19
20	7 feet 6 inches under 12-foot lanes	0·90	1·08
30		0·90	1·07
40		0·87	1·13
50		0·87	1·13
60		0·95	1·15
75		1·00	1·14
20	10 feet under 12-foot lanes	0·83	0·99
30		0·82	0·98
40		0·80	1·03
50		0·79	1·03
60		0·88	1·06
75		0·92	1·05
20	One lane of vehicles compared with one lane of U.D.L. and knife-edge loading	0·63	0·75
30		0·62	0·74
40		0·60	0·78
50		0·60	0·78
60		0·66	0·80
75		0·70	0·80

No allowance has been made for impact in these comparisons. Recent investigations [4] seem to indicate that this is not likely to amount to more than 25 per cent, nor to affect more than two adjacent wheels or a single axle; that is, the moments for the vehicles might be increased by a maximum of less than half span in foot-tons, and the shears by less than 2 tons. The amounts are negligible within the range.

For spans in excess of 75 feet, as has been suggested earlier, it seems reasonable to derive an equivalent uniformly distributed live load (taken in conjunction with a knife-edge load of 27,000 lb. per lane) simply by equating the moments and shears per lane of vehicles with the corresponding effects under the distributed load.

In *Figs 4* proposed values are shown, based on the train of traffic illustrated in *Figs 3*. There is not a great deal of difference, in this range, between the equivalents for moment and shear; the curve shown has simply been drawn smoothly, using the maximum values as a guide, and

Figs 4



	SLABS	BEAMS
Lane width 10'-0" or less	Lane loading = loading from *Fig. 1* × *W*/10 Load per sq. ft = lane loading from *Fig. 1* × 0·1 Knife-edge load per foot = 2,700 lb.	
Lane width over 10'-0"	As above	Lane loading = loading from *Fig. 1* Load per sq. ft = lane loading from *Fig. 1* × 0·1 ÷ *W* Knife-edge load per foot = 27,000 ÷ *W*

W = width of lane

Proposed Normal Equivalent Uniformly Distributed Lane Loading
(to be taken in conjunction with a knife-edge load of 27,000 lb. per lane)

marrying it to the 2,200 lb. per linear foot value already adopted at 75-foot span.

There is obviously room for a good deal of speculation about the concentrations and arrangements of traffic assumed for longer spans, so that any arrangements adopted should be looked on mainly as a guide. It is

worth noting that fairly considerable variations in the assumed traffic distribution do not have such a marked effect on the equivalent distributed load. Moreover, quite large differences in live load are not, in fact, of great significance. Thus, for spans of, say, 300 feet, the dead-to-live-load ratio will be approximately 3 to 1. Consequently an error of 25 per cent in the estimation of the actual live load represents an error of only about 6 per cent in total load.

Short-Span Members and Units Supporting Small Areas of Deck

For various reasons, it is desirable that these smaller members should be designed to carry a more severe concentration of load than is provided by the single wheel or axle of a 22-ton lorry. There is the possibility of oddities in design of vehicles which, for one reason or another, are permitted to use the highways freely. Such vehicles would not have a significant effect on larger members, but might seriously overstress the smaller. Contributing to the severity of this, is the fact that, in small members, the load will be applied quite rapidly, whilst comparatively small irregularities in road surface may momentarily transfer a quite large proportion of the total load of any axle to one wheel. It is difficult to suggest a suitable criterion for these members, but it would seem satisfactory to adopt something of approximately the same weight as the basic heavy wheel of the former trains, that is to say, 7 to 7½ tons. A heavy steam-roller can, for instance, have a wheel about this weight and the total weight of a "legal" axle is also approximately equal to it. Adding an allowance of 25 per cent for impact, this brings the total wheel weight to, say, 9 tons, and it would seem suitable to use two of these wheels, 3 feet apart as a basis of design for small units of deck.

Since, in loaded lengths of less than about 20 feet for longitudinal members, these two 9-ton wheels will have a more severe effect than the 22-ton lorries, the distributed load for the proposed design loading has been based on them, being married to the horizontal portion at 20-foot span. Table 2 gives a comparison with the assumed distributed loads (shown in *Figs 4*), from which it will be seen that there is a close identity in both bending and shear.

The main usefulness and necessity for two-point loads such as these is in designing small units of deck which would be fully occupied by one or two wheels, such as, for instance, short cantilever projections, small members of battle-deck construction, and so on. It would be difficult to formulate a uniformly distributed load which would meet all cases of this sort, and would, in fact, be an unnecessary complication, since one or two point loads are at least as simple, and probably simpler, to apply.

Later, reference is made to the four wheels comprising an axle of abnormal loading. These wheels weigh 11¼ tons for the heaviest of

abnormal loads and, when using them, 25 per cent overstress on dead and live load is suggested as a reasonable relaxation. In the type of member considered, the dead load is small, and can be neglected without making

TABLE 2.—SHORT SPAN MEMBERS

Ratio : $\dfrac{\text{effects of } 2 \times 9 \text{ ton wheels}}{\text{effects of proposed design loading}}$ on longitudinal members up to 20-foot span.

Span : feet	Bending Spacing			Shear Spacing		
	5 feet ($\frac{1}{2}$ of 10-foot lane)	6 feet ($\frac{1}{2}$ of 12-foot lane)	7 feet 6 inches on 12-foot lanes	5 feet ($\frac{1}{2}$ of 10-foot lane)	6 feet ($\frac{1}{2}$ of 12-foot lane)	7 feet 6 inches on 12-foot lanes
4	0·92		0·85	0·925		0·85
6	1·01		0·92	1·01		0·92
8	1·01		0·92	1·01		0·92
10	1·01		0·92	1·01		0·92
12	1·0	1·07	0·92	1·0	1·07	0·92
14	1·0	1·06	0·91	1·0	1·07	0·91
16	1·02	1·09	0·93	1·02	1·09	0·93
18	1·06	1·14	0·98	1·07	1·14	0·97
20	1·14	1·22	1·06	1·15	1·23	1·05

much difference to the result. If, therefore, the wheels which form the basis for design of small units are taken as $11\frac{1}{4}$ tons, with a permitted overstress of 25 per cent the result will be almost the same as for the 9-ton wheels.

The circumstance is fortuitous, but the adoption of this expedient will simplify the statement of loading regulations, in so far as it makes one part of the loading serve two purposes.

CROSS-GIRDERS UNDER NORMAL LOADING

Table 3 shows a comparison between the effects of vehicles and the proposed design loading derived for longitudinal beams on cross-girders carrying two, three, and four lanes of traffic. As before, a knife-edge load of 27,000 lb. per carriageway lane, lying across the bridge, has been used in conjunction with the distributed load.

The assumptions made are the same as for longitudinal members, that is, that there will be only one 22-ton lorry per lane width, but that the adjacent wheels of two of these can be at 3-foot centres. Where there are more than two lanes on the cross-girder, additional vehicles weighing 11 tons each have been assumed, one being added for each additional lane. Longitudinally, the same disposition of traffic as for longitudinal beams has

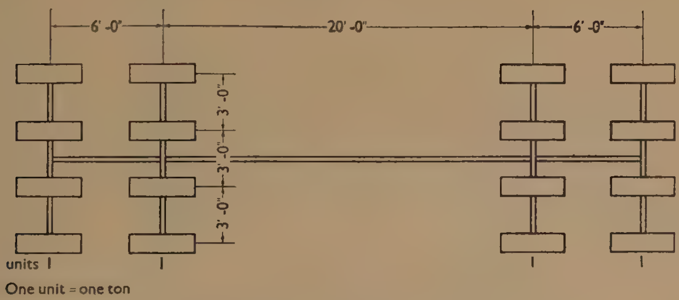
been adopted. The total design loading has been taken as spread over the full width of lanes ; for this purpose, it has simply been assumed that there are lanes lying parallel to the cross-girders, having the same width as those derived for the width of carriageway on the bridge.

Although a somewhat artificial conception, the various values of the ratio, effects of actual vehicles ÷ effects of design loading, given in the Table lie within a limited range, and so justify its adoption. The ratio falls somewhat below unity, showing that cross-girders designed on the basis suggested are capable of carrying at least 25 per cent greater actual loading than has been considered. This is ample to cover possible impact effects and to leave a reserve for possible greater concentrations of load.

Abnormal Loads on Bridges Designed for Normal Loading

The arrangement of wheels and axles proposed as a basis for abnormal loading is shown in *Fig. 5*. Any bridge designed to carry normal loading

Fig. 5



Abnormal Unit Loading Train

will be capable of taking a certain number of units of abnormal loading. Since abnormal loading consists of two concentrations on limited areas, this number will vary, especially in medium- and short-span structures, according to the spacing and arrangement of beams.

Whilst, for main roads, it will often be desirable that bridges should be able to carry 30 units of such loading, this will not always be the case. It would be uneconomical to make an envelope-curve for this intensity which would cover all configurations of deck. Moreover, since for abnormal loading, 25 per cent overstress is later suggested as permissible on dead and live load, estimation of an equivalent uniformly distributed load would involve taking dead load into consideration. Further, since the abnormal load is of limited width, 10 feet, it lends itself particularly to the use of these recent developments in structural theory considered by Guyon,[5] Massonet,[6] Thomas and Short,[7] and others. These studies relate to the

redistribution of load between beams which takes place as a result of the stiffness of interconnecting parts, such as a deck slab or diaphragms; since these properties vary from deck to deck they cannot be taken into consideration fully and satisfactorily in estimating an equivalent design loading.

It can be assumed, however, that where the dead load is similar to that of a normal reinforced-concrete deck, all beams designed to carry the full

TABLE 3.—CROSS-GIRDERS

$$\frac{\text{Effects of normal vehicles}}{\text{Effects of proposed design loading}}$$ on cross-members supporting 2, 3, and 4 carriageway lanes

Span: feet	No. of lanes	Width of lanes: feet	Spacing: feet	Ratio for bending	Ratio for shear
20	2	10	5	0·64	0·71
22	2	11	5	0·68	0·74
24	2	12	5	0·72	0·78
30	3	10	5	0·65	0·66
33	3	11	5	0·68	0·69
36	3	12	5	0·71	0·72
40	4	10	5	0·62	0·64
44	4	11	5	0·64	0·67
48	4	12	5	0·67	0·70
20	2	10	7·5	0·69	0·77
22	2	11	7·5	0·73	0·81
24	2	12	7·5	0·78	0·85
30	3	10	7·5	0·69	0·72
33	3	11	7·5	0·74	0·75
36	3	12	7·5	0·77	0·78
40	4	10	7·5	0·67	0·70
44	4	11	7·5	0·70	0·73
48	4	12	7·5	0·73	0·75
20	2	10	10·0	0·70	0·78
22	2	11	10·0	0·74	0·82
24	2	12	10·0	0·79	0·86
30	3	10	10·0	0·71	0·73
33	3	11	10·0	0·75	0·76
36	3	12	10·0	0·78	0·79
40	4	10	10·0	0·69	0·71
44	4	11	10·0	0·71	0·74
48	4	12	10·0	0·74	0·77
20	2	10	30·0	0·74	0·82
22	2	11	30·0	0·78	0·86
24	2	12	30·0	0·83	0·90
30	3	10	30·0	0·74	0·76
33	3	11	30·0	0·78	0·80
36	3	12	30·0	0·82	0·83
40	4	10	30·0	0·72	0·74
44	4	11	30·0	0·74	0·77
48	4	12	30·0	0·78	0·81

normal loading proposed will also be able to carry at least 20 units of abnormal loading, allowing reasonable overstress, whilst members having a span of over 100 feet will generally be adequate for 30 units.

In long-span bridges, the effects of normal traffic will exceed those of a 30-unit abnormal load, but in the medium and short range, economy can be gained by using this, or any other number of units as a check loading. Considerable flexibility will also be gained in this way, since reduced intensities of normal loading can be combined with any number of units of abnormal loading as local conditions require. This is of particular value from the international point of view, that is, so far as B.S.153 is concerned.

In this respect, there is further ground for the adoption of the pair of heavier 9-ton wheels as a basis for the design loading used for small units and short-span beams and stringers. These usually occur in large bridges where the main members, designed to carry several normal vehicles, can also carry quite large abnormal loads without further consideration. If, however, the small units of these bridges, as, for instance, short stringers, were designed to a strength only sufficient to carry one or two wheels of a normal vehicle, they would limit the total strength of the bridge to this amount, so that the design would be "unbalanced."

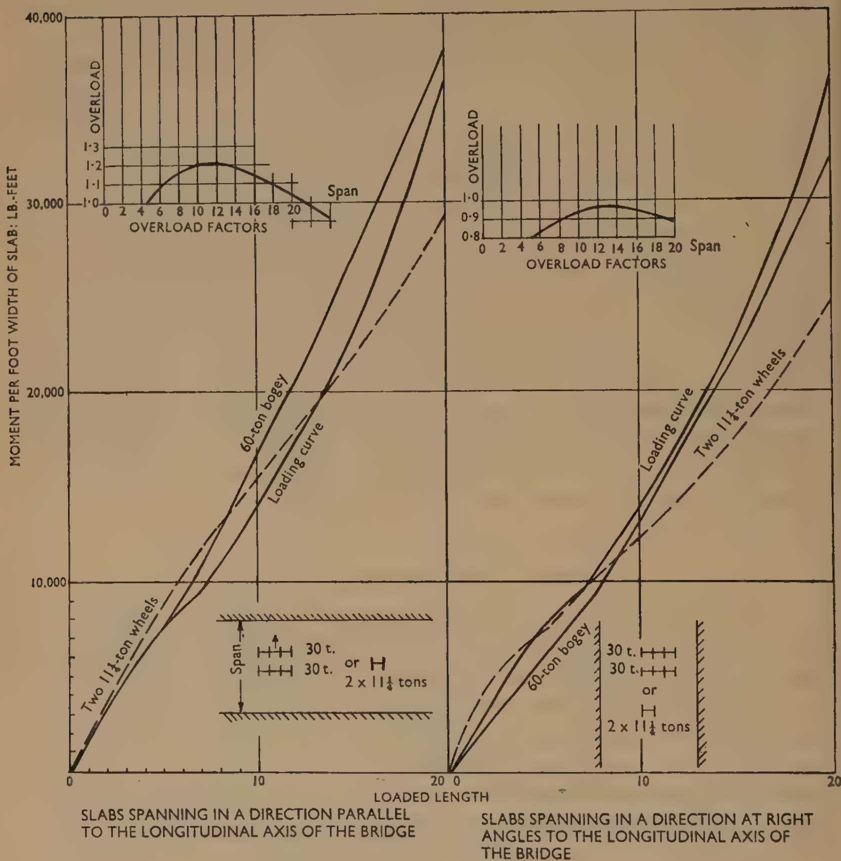
Reinforced-Concrete Slabs

As for short-span beams, oddities of vehicle design must be considered in relation to reinforced-concrete slabs. An approximately correct analysis of a reinforced-concrete slab under the effect of groups of wheels is a laborious process, whilst the saving in materials for reductions in total load is small in comparison with the decrease in load. Reinforced-concrete-slab bridges are also of such frequent occurrence that it seems both unnecessary and undesirable to treat them in the same way as beams, that is, to design for normal traffic and subsequently check for abnormal traffic. There is, too, the additional reason that once a slab is built, it is not always easy to ascertain how it was reinforced, if and when its strength is to be assessed.

It was, therefore, considered that a suitable basis for normal design loads for slabs would be to relate these to a two-axle bogey weighing 60 tons (the weight of a bogey of 30 units of abnormal loading), each axle having four wheels, and also to take into consideration, on the smaller spans, the two $11\frac{1}{4}$-ton wheels suggested as a basis for the design of small units.

The spacing of the wheels on the axles of the bogey has been taken as 3 feet, and the axles have been assumed to be 4 feet apart. This latter dimension compares closely with vehicles in use today to carry 100 tons gross weight. The majority of heavy abnormal loads travelling in Britain are of approximately this weight; some allowance is necessary to compensate for faulty estimation of total load, and for inaccuracies in placing

Fig. 6 *Fig. 7*



NORMAL LOADING ON REINFORCED-CONCRETE SLABS

the load centrally on the trailer. To allow for these errors, the bogey weight has been increased from 50 tons nominal value by 20 per cent to 60 tons.

An equivalent uniformly distributed load, taken in conjunction with a knife-edge load of 27,000 lb. per lane is shown in *Fig. 4*, whilst *Figs 6* and *7* show curves for bending moment for the bogey, the two $11\frac{1}{4}$-ton wheels, and the equivalent loading.

Since dead load can be accurately estimated for reinforced-concrete slabs, this has been taken into consideration in *Figs 6* and *7*, with each of the loadings, the amount of dead load taken being that appropriate to the design loading. Because the loadings considered are exceptional, it is reasonable to permit some degree of overstress, provided this does

not exceed 25 per cent, the amount which is later suggested as satisfactory in dealing with abnormal loading. These overstresses are plotted on the same diagrams as overload factors.

In *Figs 4* it will be seen that as the span of the slabs increases to 20 feet, so the distributed load converges on 2,200 lb. per foot run of lane; this amount, satisfactory for beams of more than 20-foot span, is also suitable for larger-span slabs, so that it is convenient to merge the two curves at this point.

Method of Estimating Moments in Slabs

The method adopted in estimating the bending moments under the bogey and the two $11\frac{1}{4}$-ton wheels was derived from an approximate method[8] based on the more rigorous analysis of Westergaard.[9]

The bending moment in the direction of the span from a single wheel placed at the centre of an infinitely wide slab was taken to be:

$$M_{ox} = \frac{PS}{2{\cdot}32S + 8c}$$

where M_{ox} denotes the principal moment per unit width of slab,

P ,, the weight of the wheel,
S ,, the span of the slab, and
c ,, the diameter of the contact area of the wheel.

Continuing to consider the slab as infinitely wide, the increase in M_{ox} from another wheel on the same transverse section is given by Westergaard as:

$$M_x = 0{\cdot}21072\,P\left(\log_{10}\coth\frac{\pi y}{2S} + \frac{1{\cdot}0085y}{S\sinh\dfrac{\pi y}{S}}\right)$$

to which equation a reasonable approximation is given by:

$$M_x = \frac{M_{ox}}{1 + 10(y/S)^2}$$

where y denotes the distance between centres of wheels.

These equations provide a reasonably accurate estimate of the bending moment under any one wheel from the effects of the four wheels of an axle.

There remains to be taken into consideration the effect of the second axle of the bogey. Westergaard has given an equation for the maximum moment under a pair of wheels lying on a line parallel to the span as:

$$M_{max} = M_{ox} + 0{\cdot}21072\,\mathrm{P}\,\log_{10}\tfrac{1}{2}\cot\frac{\pi a}{4S}$$

where M_{ox} denotes moment for one wheel placed at the point of maximum moment, and a denotes distance between centres of wheels.

It was assumed that the proportionate increase in total axle-moment

from an equal following axle would be the same as the proportionate increase in wheel-moment from an equal following wheel.

This last assumption is not quite true, nor are the principal moments exactly parallel to the axes of the slab ; the approximation does not involve serious error, however, and what error there is, is on the safe side.

When slabs spanning in a direction transversely to the direction of traffic were investigated, the procedure was modified accordingly.

From the form of the equations, the moments, and consequently, the distributed loads were derived for strips of slab of unit width. It will be clear from the assumption of a slab of infinite width, and from the further assumption that no other load is on the slab at the same time, that the conception of lane widths is irrelevant so far as slabs are concerned. Consequently, variation of the intensity of loading arising from variations of width of lane, as is considered suitable for beams, would be inappropriate. The distributed load for slabs, shown in *Figs 4*, has been given for a 10-foot-wide lane ; for wider lanes it should, therefore, be varied according to the width of the lane *pro rata*, so that the intensity per square foot remains unaffected.

Loads Running Near the Edge of Slabs

These comparisons have been made for slabs of infinite width and for wheels placed near the centre of the slab. There remains the possibility of a load running near to the edge of the slab, thus increasing the actual moments under the vehicle. It would be uneconomic to deal with this by making the whole slab of a sufficient strength to support extreme eccentric loading conditions. The simpler expedient is to provide some regulation which calls for a stiffened edge, or requires the slab to be extended some way beyond the side of the carriageway.

When a single-point load is placed centrally on a wide slab, the reaction per unit width of slab is a maximum at a point on the support in line with the load, and diminishes as the transverse distance from this point increases, becoming zero at approximately 0·8 × span transversely from the load. When the edge of the slab lies within this distance, the reactions per unit length of support will be increased, as will the moments in the elements of the slab.

The introduction of an edge beam will modify the distribution of the reactions, relieving the slab of a proportion of these. If it is of adequate stiffness, the conditions in the slab will be as if the load were placed centrally on a slab of infinite width.

When a point load is placed over the edge stiffening beam, the reaction at the end of that beam with a load W at midspan will be, fairly closely, $\frac{1}{2}W$; the actual reaction will vary according to the relative stiffnesses of beam and slab, but the approximation is on the safe side. It can also be

assumed that there is no great degree of dispersion of the load along the beam.

Where the load is removed from the edge of the slab by a distance x, a reasonable approximation [8] to the load carried by the beam is

$$W = P - 0{\cdot}68\, r\,(S + x)$$

where W denotes load on beam, P the wheel load, S the span, x the distance of wheel from edge, and r the reaction on unit length of slab.

$$\text{Approximately, } r \text{ is } \frac{P}{S} \times \frac{\frac{x}{S}}{\left(0{\cdot}164 \times 0{\cdot}68\frac{x}{S} + 0{\cdot}52\left[\frac{x}{S}\right]^2\right)}$$

This load is distributed along the beam, with its maximum intensity in line with the wheel, and diminishing rapidly away from this point.

From the foregoing it is possible to make an approximation of the amount of load taken by an edge beam, and to state requirements for its strength.

For longitudinal slabs it is most convenient to give an equivalent width of design loading to be carried by the edge stiffening, the amount required being equal to one-quarter of the span. In suggesting this, it has been assumed that it is most unlikely that the wheels of a heavy vehicle will ever be at the extreme edge of the carriageway. The edge stiffening could be provided, either as an additional width of slab extending beyond the kerb for a distance equal to $\frac{1}{4}$ span, or alternatively, advantage could be taken of the extra depth available immediately behind the kerb to make the provision against edge loading narrower and deeper. The additional width need not be greater than 7 feet 6 inches and should not be less than 2 feet.

It will be appreciated that these provisions will effectively prohibit the formation of free longitudinal joints along the carriageway in simple slab bridges. Such joints are undesirable in any part of a structure where transverse distribution has been assumed to take place, since the distribution must stop at a free joint.

In transverse slabs, conditions are rather different. If unsupported edges of slabs are formed, they will lie across the carriageway, and will inevitably be loaded with the maximum wheel loading of one axle of the bogey at the extreme edge, together with a considerable proportion of the following axle.

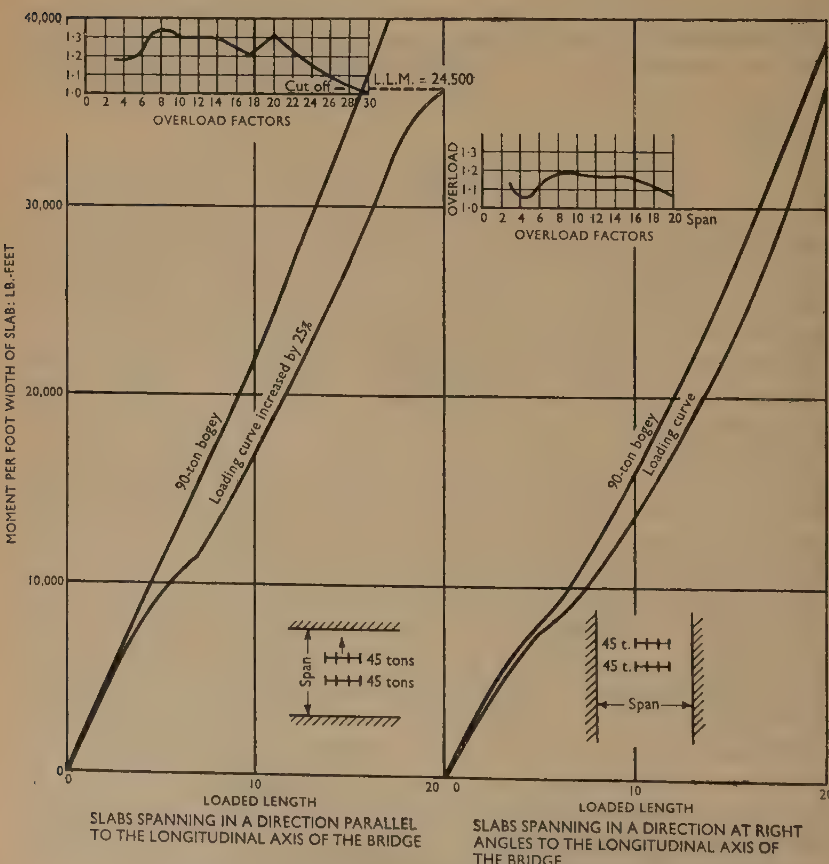
Where such edges on transverse slabs occur, they should be supported by properly formed beams, having a depth in excess of the slab. As an approximation, these beams could be designed to carry the two $11\frac{1}{4}$-ton wheels at 3-foot centres, suggested elsewhere as an alternative loading, or the edges could be supported on curtain walls built up from the abutments.

Maximum Abnormal Loading on Slabs

The weights and spacing of the wheels, axles, and bogeys of the proposed abnormal loading train (discussed later) are shown in *Fig. 5*. Based on the same method as was set out earlier, the moments per unit width of slab have been calculated for 45 units of this loading, and are plotted in *Figs 8* and *9* respectively, for slabs spanning longitudinally and transversely. In *Fig. 8*, the moments caused by a simple adaptation of the equivalent loading derived for normal conditions have been plotted.

For longitudinal slabs of less than 20-foot span, the requisite strength, allowing a maximum overload of approximately 30 per cent on dead and live load, has been gained by increasing the suggested normal loading-

Fig. 8 *Fig. 9*



Abnormal Loading on Reinforced-Concrete Slabs

curve moments by 25 per cent, and the dead load commensurately to carry this increase in moment. When the span is equal to or greater than 20 feet, the proposed normal loading-curve moments will suffice without increase.

Some awkwardness arises at the change point from about 17-foot-6-inch span to 20-foot span when the proposed normal design loading, increased by 25 per cent, is altered to the normal loading alone. This can be dealt with by making the live load moment for a 20-foot span the maximum for spans below this length. The effect is to introduce a rapidly diminishing percentage increase in design loading over a very short range.

In transverse slabs, the effects of the 90-ton bogey of the abnormal load do not exceed by more than 20 per cent those of the suggested normal loading-curve moments throughout the range of spans, so that design on the latter basis would suffice without increasing the live loading.

Abnormal Indivisible Loads

Abnormal indivisible loads are fairly strictly controlled, so far as load transmitted to the road surface is concerned, by a Statutory Instrument.[3] Partly as a result of concern for the safety of bridges, these new regulations limit the total weight which can be carried on any one axle to 45 tons; the weight carried by the following axle is controlled by its distance from the first axle. Since the axles of any bogey have some sort of balancing mechanism between them, the loads on each axle will be the same; thus the regulation effectively limits the weight on either axle and the total weight on a two-axle bogey on the basis of the spacing of the axles. The maximum permitted load is 90 tons for any two-axle arrangement when these are placed 7 feet apart. Similarly, the distance between bogeys is controlled by limiting the load carried on additional axles as a function of the distance apart. Such axles must have at least four wheels, since the regulation limits the maximum wheel load to $11\frac{1}{4}$ tons.

The upper limit of gross weight for the whole vehicle is 150 tons, but it is possible, by obtaining a special order, for a haulier to be permitted to move heavier gross loads. Since the issue of this order is under the control of the Minister of Transport, steps can be taken to ensure that the worst effects of the specified maximum vehicle are not exceeded.

There is, therefore, a precisely defined "heaviest" abnormal vehicle. Since there is no likelihood of these vehicles coming in train, or passing each other on a bridge (there is legislation to prohibit this), the simplest and most convenient representation for design purposes is to use a train of four axles, each with four wheels, which will conform closely to the permitted maximum vehicle.

Fig. 5 illustrates such a representation, stated in terms of unit axles, which it is considered would be suitable for adoption. Forty-five units of

this design load can be taken as equivalent to the maximum permitted weight of vehicle.

No allowance need to be made for impact, since these vehicles are so slow moving that, for conditions on British roads, this effect is unlikely to be material. It is also considered that no allowance need be made for other vehicles in the same carriageway lane, since it is unlikely that lighter vehicles will be in a position to have much material effect. Heavy normal traffic is also most unlikely to be crowded in other lanes on a bridge when abnormal loads are crossing; it is thought that it would be a reasonable and safe assumption to consider only one-third normal loading as being in other traffic lanes at the same time as the abnormal train is on the bridge.

A permissible overstress of 25 per cent on dead and live load when designing for abnormal loads seems a reasonable relaxation; the vehicle is sufficiently accurately known in its weight and configuration, and so uncommon in its occurrence as to justify this. Because of the close identity between assumed and actual vehicles, it is incumbent on the designer to take into consideration every legitimate factor which will bring about dispersal or redistribution of load to the member under consideration in order to achieve economy in materials. Thus, for instance, where a grillage-type deck consisting of beams and transverse interconnecting diaphragms or deck slab is being considered, it is desirable that the redistribution of loading between the beams be taken into consideration. On the other hand, the same identity between design and actual loading requires that any assumptions made in calculation must assuredly be reflected in the structure.

Reduced Intensities of Abnormal Loading and Assessment of Bridge Strengths for Specific Vehicles

The Statutory Instrument [3] governing the design of vehicles carrying abnormal indivisible loads has probably gone unnoticed in the structural engineering world, but nevertheless it represents a step of major importance in bridge design. In this respect, however, it has certain limitations; its aim was to control maximum severities of loading, and it does not provide intermediate classifications of loads which are, in proportion to their gross weight, less severe than the maximum.

As an extreme case, a gross 90-ton load could be carried on two 45-ton axles at 7-foot centres, so that, on short-span bridges the effects would be as severe as 45 units of the proposed abnormal design loading. Also, as the load on a pair of axles is diminished, so the regulations permit a reduction in the axle spacing which, to some extent offsets the benefit of lightening the load.

The proposed abnormal design load has, however, fixed axle-centres, irrespective of the number of units taken, so that the effects of this loading vary directly with this number. This is desirable so far as the design of

bridges and the assessment of bridge strengths is concerned, since it gives a uniform range of carrying capacity, varying directly with the number of units of design load assumed. There remains, however, the necessity to relate actual abnormal vehicles of intermediate weight to an equivalent number of units of the abnormal design loading. Thus, in the selection of the number of units to be used in design, due regard must be paid to the dimensions as well as the weights of the vehicles which will be travelling over the bridge. In Britain, it is considered that a general intermediate standard for main roads would be 30 units.

The Ministry of Transport and other bridge authorities throughout the country are faced with the considerable problem of estimating the capacity of existing bridges to carry particular vehicles and loads. Few of these bridges have been designed for abnormal loading, and a great number for no known loading, because they were built prior to general acceptance of a standard for design. In dealing with these, it is suggested that the simplest and most useful expedient, in the long run, is to assess the capacity of the bridge in terms of units of abnormal design loading. Parallel to this, characteristic bending moment and shear curves can be made for each type of vehicle used to carry abnormal indivisible loads; these curves could quickly be compared with a family of abnormal design loading curves to give a classification for the vehicle in the same terms as for bridges. Such a method of classification would permit ready adjustment according to the total load carried by any particular vehicle, and it is thought that the number of types of vehicle in existence is not so great as to make such a task insurmountable.

It is, perhaps, fitting to point out that a variation in the axle spacing of the design loading (an adjustment which at first sight may seem to simplify comparison) would not, in fact, contribute much to this end; the lighter types of vehicle used do not conform so closely to a standard as to make this step generally satisfactory, and it would still be necessary to compare many vehicles to a norm.

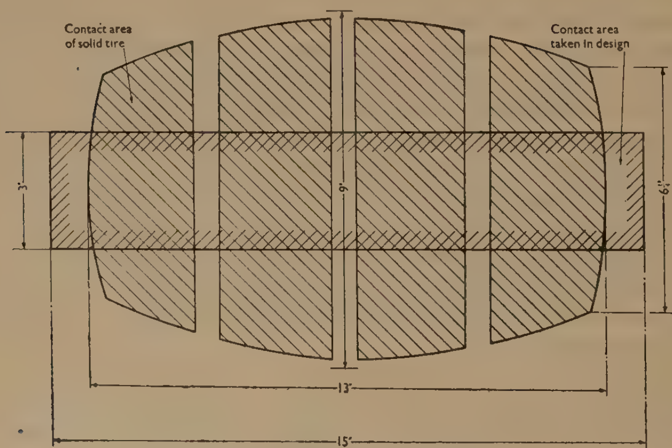
At present, very few of the vehicles carrying abnormal loads have pneumatic tires. The trend appears to be to adopt these in new designs, and, when this is done, the minimum spacing of the wheels will, of necessity, be approximately 3 feet. In the solid-tired vehicles used at present, the wheel spacings are somewhat less, being frequently not much more than the 2 feet apart which is necessary in order that the wheels be considered as separate and permitted to carry up to $11\frac{1}{4}$ tons each. In view of the trend in vehicle design, however, it is felt that this need not be taken into consideration in the design loading, provided due regard is given to it in the assessment of the strength of bridges for specific vehicles. To this end, the closer wheel-spacings can be taken into consideration in classifying any particular vehicle.

Contact-Area of Wheels

When it becomes necessary to take the contact-areas of wheels into consideration in designing members, that is, primarily in very small deck units, the difference between those for solid tires and those for pneumatic tires is so great that it would be unwise to anticipate the universal adoption of the latter. This step would have to await the time when present day solid-tired wheels have become obsolete.

Fig. 10 shows in outline the contact-area of a single solid rubber tire, loaded to 11¼ tons. On this has been superimposed a contact-area of 15 inches by 3 inches. Bearing in mind that the intensity of pressure will

Fig. 10



Contact Area of 11¼-ton Wheel

be almost constant in a direction parallel to the axle, and will diminish from the maximum at the centre to zero at the edges in a direction at right angles to this, it will be seen that this latter area gives a fairly close approximation to the severest actual conditions.

Summary of Proposals

General

Two design loadings should be adopted, one representing normal traffic, and the other, which should be a check loading, representing abnormal traffic.

Normal Loading

(1) Full normal loading should be used in any two traffic lanes, the remaining lanes being taken as carrying half loading.

(2) Normal loading should be represented by a simple distributed load per lane of traffic, varying with the span, but for beams, remaining constant for lane widths of 10 feet to 12 feet, and reduced *pro rata* for lane widths of less than 10 feet. The lane loading can be spread uniformly over the width of the lane. For slabs, the lane loading should be varied according to the width *pro rata* of the lane; that is to say, the load per square foot should not be varied according to the width of the slab.

(3) For normal traffic, the present Ministry of Transport Loading Curve, in the range from 20 to 75 feet, taken with its knife-edge load provides a satisfactory basis, using a 10-foot width of this loading to represent the lane loading.

(4) For members supporting small elements of deck, two 9-ton wheels, 3 feet apart provide a suitable basis for design. In order to conform to the maximum abnormal load wheels, they can be stated as two $11\frac{1}{4}$-ton wheels with a permissible overstress of 25 per cent. The two 9-ton wheels also provide a suitable basis for the derivation of a distributed load for beams of less than 20 feet loaded length.

(5) For various reasons, the distributed loading for reinforced concrete slabs should be based on a 60-ton eight-wheel bogey. In shorter spans, the moments for this compare closely with the two wheels mentioned above.

(6) The loading curve shown in *Figs 4* provides suitable values for distributed load to be taken in conjunction with a knife-edge load of 27,000 lb. per lane for all spans.

Abnormal Loading

(7) The train of axles shown in *Fig. 5* provides a satisfactory basis for the design of bridges intended to carry abnormal indivisible loads. In Britain, 45 units of this train should be used for bridges carrying the heaviest loads. On other main roads, 20 to 30 units might be used.

(8) Because of the fairly rare incidence of abnormal loads and the reasonably accurate knowledge of their weight and configuration, 25 per cent overstress can be allowed on both dead and live load.

(9) For the heavier abnormal loads, say 20 to 45 units, no other load need be taken on the same carriageway lane, and comparatively light loadings, say $\frac{1}{3}$ normal loadings, on other lanes.

(10) Reinforced-concrete slabs can be designed for 45 units of abnormal loading, simply by increasing the proposed normal design loading by 25 per cent for spans up to 20 feet lying parallel to the traffic. For greater spans and transverse slabs, normal design loading will suffice.

Assessment of strength of existing bridges for abnormal loads

(11) The strength of any existing bridge could be estimated, where necessary, in terms of the maximum number of units of abnormal loading which it can carry. It is suggested that this might also be done for all new bridges as they are built.

(12) Actual vehicles carrying abnormal indivisible loads should also be classified in terms of an equivalent number of units of abnormal design loading. This can be done fairly simply by plotting bending moment and shear curves for the actual vehicle, and comparing these with master curves for varying numbers of units of abnormal design loading.

Acknowledgements

The Author would like to take this opportunity of expressing his thanks for the advice and help he has had in preparing the Paper. He is indebted to colleagues in the Ministry of Transport, and to the Members of the British Standards Institution committee now revising B.S. 153; their comments and discussion while considering the problems of highway loading have contributed largely to the substance of the Paper.

In particular he wishes to thank Mr C. S. Chettoe, B.Sc.(Eng.), M.I.C.E., members of the Drafting Panel of I.S.E. 55 of British Standards Institution, and Messrs Ford and Drummond, A.MM.I.C.E.

The Paper is accompanied by eight sheets of diagrams, from which the Figures in the text have been prepared.

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Discussion

The Author, in presenting his Paper, included additional material and anticipated some points raised in the discussion.

He said that the main difficulties in prescribing a design loading for highway bridges were the wide variety and random occurrence of vehicles which had to be taken into consideration.

Whilst the heaviest wheel permitted for normal-load vehicles was 4 tons, there were locomotives permitted to have wheel loads of $8\frac{1}{4}$ tons, and various other vehicles in the light abnormal load class which might have wheels of similar weight.

Considering normal loading, it was true to say that the combinations of 22-ton, 10-ton, and 5-ton lorries used as a basis were heavy. Nevertheless they were not so severe as to be beyond the bounds of reasonable possibility. In considering them, it should not be overlooked that overloading of lorries was not uncommon. Thus, whilst 4-axle lorries should not exceed 22 tons gross weight, at least one make of vehicle of that type had been described as a 25 tonner, and actual gross weights might reach 30 tons.

Some consideration should also be given to future conditions. Bridges built under current regulations would be expected to carry traffic of the day for at least 80 to 100 years. In that time it was likely that there would be demands for freedom to operate bigger and heavier vehicles without restriction. Already, indeed, it was being proposed that the permitted weight for normal load carrying vehicles should be increased from 22 tons to 24 tons, whilst it was suggested that locomotives should be allowed up to 30 tons.

In a recent Paper presented to the Institution,[7] the problem of redistribution of load to unloaded members of a bridge deck was discussed and the limitations to the use of a distributed load in relation to that kind of analysis made clear. In that respect it might be felt that the adoption of a single train of vehicles for design purposes would permit full advantage to be taken of the latest advances in that field.

For normal loading that might be satisfactory if the weights and dimensions of vehicles conformed fairly closely to some common norm, and if their transverse distribution on the carriageway was at all precise. In reality, however, there was a random arrangement of vehicles of fairly widely differing weights and sizes. To stipulate a train of vehicles and to apply those methods, so far as heavy normal traffic was concerned, would, it was thought, be to pretend to an accuracy of knowledge of weight and arrangement that was without justification.

On the other hand, the weight and dimensions of the heavier type of abnormal load were very much more precisely known quantities; it seemed not only reasonable, but desirable to take advantage of every

factor which would diminish the effects of those vehicles on individual members.

Similarly, when considering normal loading on wide bridges it would seem legitimate to take into consideration any redistribution from the members carrying fully loaded carriageway lanes under reduced load. The width of distributed loading involved by taking two fully loaded lanes should be sufficiently great to offset local variations within the width of the lanes, whilst the assumption that the other lanes carried a reduced loading would prevent overestimation of the assistance provided by the more lightly loaded members.

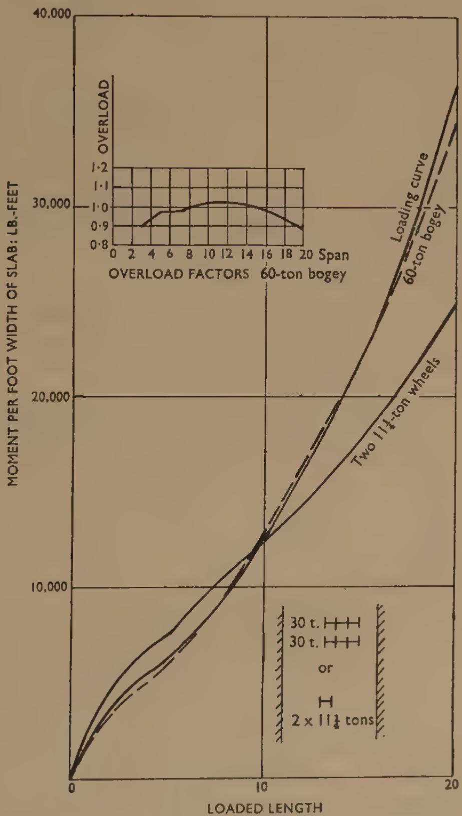
The amount of redistribution of loading from member to member from the effects of transverse stiffness became a matter of considerable importance in dealing with a type of deck now commonly built in prestressed concrete. That consisted of a series of beams, placed side by side, with the void between filled with cement-mortar, the whole system being transversely stressed. Designers generally treated that laminated slab deck as a unit with the same properties in both directions. In so far as there was sufficient transverse stressing to justify that assumption, it was legitimate. Recent experiments seemed to indicate, however, that the transverse moment was greater than often anticipated, and the amount of transverse prestress applied was not always fully transferred to the deck as a whole. The consequence was that the joints between units tended to open under maximum vehicle load. Lacking adequate transverse stiffness, moment was increased in the longitudinal direction, so that serious overload might result. It should not be forgotten that the current Ministry of Transport loading and those new proposals, so far as they related to slabs, were based on the assumption that the slab behaved as if it were isotropic. If slab loading was to be used, sufficient transverse resistance to bending should be provided to ensure that that assumption was justified. It appeared as if that transverse moment might amount to about 40 to 50 per cent of the longitudinal moment in average circumstances.

Subsequently to the drafting of the Paper, it had been pointed out that economy could be gained in the design of long-span bridges as a result of quite small reductions in the dead load of slab decks. The decks of such bridges generally consisted of transverse slabs, spanning about 5 to 7 feet between longitudinal stringers. From *Fig. 7* (on p. 340) it would be seen that the effects of the 60-ton bogey fell somewhat short of those of the proposed design loading. That was a consequence of adopting the same design loading for slabs spanning transversely as was used for those spanning longitudinally.

It was obviously preferable to have as few variations of the design loading as possible. Any reduction in the weight of transverse slabs would, however, be of such material advantage in reducing the dead load on large span bridges that it seemed worth while to introduce what

amounted to a comparatively simple variation in the loading curve, that was an additional curve for loading on those slabs. That curve was shown in *Fig. 4*. It had been derived from the effects of the 60-ton bogey described in the Paper. As has been done for longitudinal slabs, the effects of the bogey and the proposed design loading had been plotted, taking the dead load of an ordinary R.C. slab into consideration. That

Fig. 11



COMPARISON OF MOMENTS ON TRANSVERSE SLABS

comparison was shown in *Fig. 11* with, in the top corner, the overload factors plotted against the span ; those fell within reasonable limits.

Mr C. S. Chettoe said that he wished to add a few words to the Author's introduction from the point of view of the Ministry of Transport. As was stated in the Paper, the British Standards Institution were revising their girder bridge specification, which included a standard highway loading, and, at the same time, the Ministry of Transport were revising

their Standard Loading Curve. For a number of years those two loadings had been similar but not exactly the same, and it was very desirable that they should be brought completely into line.

That was rather a difficult matter because the British Standard Loading should be a universal loading not solely applicable to Great Britain and because the future loading should be as simple as it could be, though it would be of much the same type as the old one.

It had been desired at the same time to consider it from first principles with a view to seeing that it did reasonably represent the traffic that might be expected on the roads at the present day. Mr Chettoe thought it had been possible to do that. The Ministry of Transport had introduced the concept of traffic lanes to get a reduction of loading on wide carriageways; they had reduced the loading quite a bit in the middle range, and had done something else to which he would make reference. All that, of course, had taken time. The original intention and hope had been that, before the Author produced his Paper, the British Standards Institution and the Ministry would have issued those loadings. But unfortunately owing to the quite considerable difficulties that had not been possible, and accordingly the Paper had been written in its existing form.

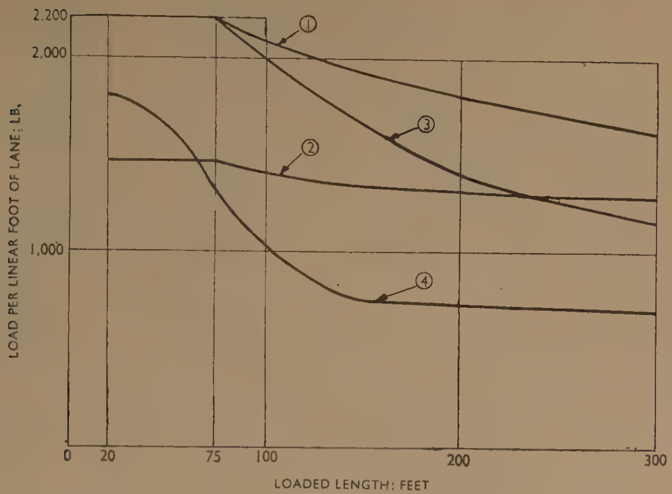
There had been one further difficulty. All the previous loadings had been related to a train of vehicles—a traction engine and a series of trailers—which was no longer a common feature on the roads. The loading had now been related, in the main, to the actual vehicles which used the road—22-ton, 19-ton, and 12-ton lorries and private cars—with a new provision for abnormal load. The Ministry of Transport and the British Standards Institution had been able to do that, whilst still maintaining the general appearance of the load.

Mr F. M. Fuller said that in *Fig. 12*, curve 1 showed the 1931 Ministry of Transport loading, curve 2 the loading without impact, curve 3 the proposed loading, and curve 4 the loading adopted in America for their important highways. It would be noticed that the advantage of the modern theory that there was practically no impact on road vehicles had been lost by an increase in the basic loading, and if that increase had occurred, of course it was only right that that should be so. He also called attention to the fact that the two loadings were constant from 20 feet to 75 feet, and that the American loading, in order to distinguish between bending moment and shear, had varying knife-edge loads.

Fig. 13 showed the bending moments attained from those loadings. Again it would be noted that from 20 feet to 75 feet the old loading and the new loading were identical. Beyond that point the new loading was slightly less than the old loading, not only from the reduction in impact but from the fact that the Author had taken a more realistic view of the dispersion of vehicles on long spans.

One point he would mention was that if the shorter spans were considered, it would be found that actually for 5-foot to 17-foot spans the new

Fig. 12



Loading	Year	Impact	Knife edge load per lane: lb.
1. Ministry of Transport . .	1931	Incl. (50% max.)	27,000
2. " " . .	1931	Exclusive	18,000
3. Proposed normal M.O.T. .	1954	Incl. (25% on 1 axle)	27,000
4. American H20–S16 . .	1954	Incl. (30% max.)	18,000 (B.M.) 26,000 (Shear)

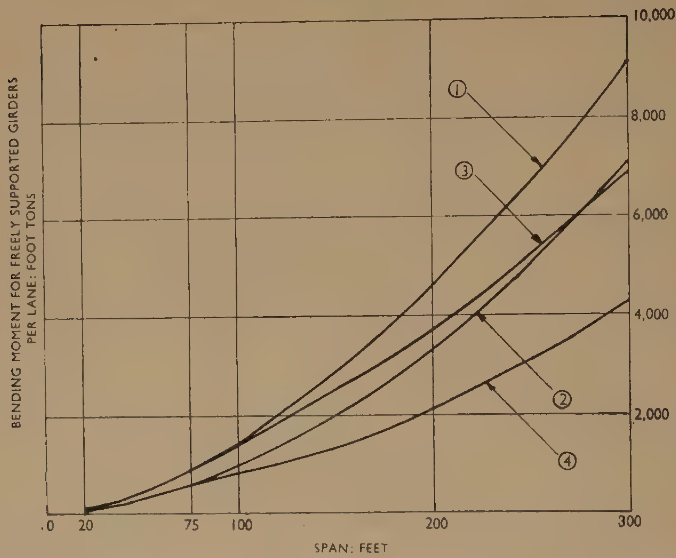
EQUIVALENT LOADING FOR HIGHWAY BRIDGES

loading was higher than the old loading and for one particular span it was actually 50 per cent higher than the old loading.

The difficulties of getting a rational and satisfactory equivalent loading for highway bridges should be appreciated by all beginners. With railways the loads were applied direct to the rails and the bending moment could be calculated very simply, but with roads not only was there a variation in the loading but there was a variation in the disposition of the members supporting the road, and any equivalent load was bound to have inaccuracies.

One very satisfactory feature of the loading in the Paper was that it was now a matter of dealing with carriageway lanes and not with widths of carriageway as with the old loading. As long ago as 1946, in a discussion on the Menai Suspension Bridge, it had been pointed out that, although at the towers the carriageway was 20 feet wide and widened out later to 23 feet, under the existing rules it was necessary to consider the full 23-foot-loading ; and he had produced figures at the time to show

Fig. 13



Loading	Year	Impact
1. Ministry of Transport . .	1931	Inclusive (50% max.)
2. " " . .	1931	Exclusive
3. Proposed Normal M.O.T. . .	1954	Inclusive (25% on 1 axle)
4. American H20–S16 . . .	1954	Inclusive (30% max.)

BENDING MOMENTS FOR HIGHWAY BRIDGE LOADINGS

that the intensity of loading on that bridge in the midst of the Isle of Anglesey was 50 per cent greater than in a bridge recently opened in the centre of New York. Fortunately the adoption of those lanes would reduce that discrepancy.

Those loadings appeared to be based on the weights of 22-ton lorries. There was a diagram of three following one another, next to three in the adjacent lane. He did not know whether anyone had ever seen six 22-ton lorries together, and whether it was quite fair to take that as the basis of a normal loading. The vehicles were referred to as " short-based vehicles " and it was assumed that they were 24 feet overall. Taking 25 feet basic length and assuming only 1 foot between vehicles would make the 24 feet overall. Were 22-ton lorries made as short as 24 feet? He did not know—and he was hoping that the Author did. He had always thought that when exceptionally short wheel-bases were adopted, there was an appreciable overhang beyond the back axle so that a longer lorry

resulted, which would naturally give more distribution; he was wondering whether it was possible to get three 22-ton lorries on a 75-foot span. He suggested that it would be more reasonable to take a 30-foot unit which would give a slight reduction.

He wished to refer to the Tables that the Author had produced, and in that connexion he thought the Author was to be congratulated on putting all his cards on the table. When the Author had made calculations to show that the wheel loads produced only 60 per cent of the moment that the equivalent load produced, he did not hesitate to say so—and he hoped that the Author would not mind his calling attention to that fact.

In Table 1 in the Paper, the variation of the ratios for moment exceeded 25 per cent and for shear exceeded 20 per cent, and the variation between the smallest ratio and the largest ratio in that Table was as much as 50 per cent. In Table 2 there were figures of 40 per cent for all three ratios, and in Table 3 there were 33 per cent, 40 per cent, and 50 per cent. Those ratios indicated the impossibility of getting a uniform loading to correspond closely with assumed wheel loadings.

The question arose whether that loading was close enough. He did not know whether it would be fair to take the lowest ratio in all those three tables and compare it with the highest ratio as a measure of—if he used the word "error," he hoped the Author would understand—as the discrepancy between the wheel loads and the uniform loads; but the lowest ratio was 0·62 and the highest ratio was 1·23, a variation of 100 per cent. The question that had to be answered was whether a curve with those variations came within the Author's dictum that the curve should cover all conditions without being uneconomical.

He wished to make a brief reference to the abnormal loading. Abnormal loading had been adopted in London for many years. A very satisfactory feature of that abnormal loading was the 6-foot spacing between the front axles, which provided a very desirable relief. Abnormal loads could cause trouble. They varied from cylinders 150 feet long to such trifles as an embalmed whale which had had to come through London.

He assured the Author, although the Author already knew that after the many discussions they had had on the Paper, that he was not seeking to be unduly critical; but having seen the Author's figures, he was very puzzled, and candidly he had not been fully aware of the complications or the possible variations in the problem. He wished to conclude by asking a few questions.

Was the Author satisfied that the curve was sufficiently accurate for the purpose when the ratio figures showed such large variations? Should they not adopt the American system of having different knife-edge loads for bending and shear? It would not complicate the calculations; he was not suggesting a variation in the spread load, but merely in the knife-edge load; it was standard practice with railway loading to have a slightly heavier load for shear than for bending; and he thought that if a

varying knife-edge load were to be adopted, more accurate results would be obtained. And lastly, did the Author consider that, with small spans less than 50 feet or 60 feet, there should be an alternative loading of wheel loads given so that those engineers who wished to do their own calculations could take advantage of the resulting economy ?

Mr O. A. Kerensky congratulated the Author on his Paper and on having been able to reduce the current Ministry of Transport loading by a considerable amount, although perhaps not yet to the final limit. He believed that the previous Ministry of Transport loading had been the heaviest in the world, and he was afraid that the loading now proposed would still have that distinction, but the Paper explained why it had to be heavy, where possible future savings and reduction of loading might be expected—perhaps after further tests and statistical observations—and how the loading itself was arrived at, which in the past had been shrouded in mystery.

With regard to possible savings in the future, the replacement of solid tires by pneumatic ones was the first point that arose. In England solid tires were still used, exerting large pressures per wheel. The diagram on p. 348 (*Fig. 10*) had been obtained from an actual tire loaded with $11\frac{1}{4}$-ton static load (9 tons + 25 per cent impact) at the Building Research Station in connexion with tests on steel battle decks to be used in long-span bridges. Whilst those loads existed and that pressure was to be provided for, the bridges would have to be designed for 9-ton wheels acting on strips 3 inches wide by 15 inches long. In America, a loading of 9·3 tons had been used until 1949, when it was reduced to 7 tons, which meant that they had been satisfied that the old-fashioned wheel had gone out of use in America. Perhaps something could be done to kill it in England, too ! He thought that if that loading was to be considered as a British Standard loading for general use, the provision for a 9-ton wheel for foreign countries and for the Dominions and Colonies was unreasonable. Most of the Colonial roads would not stand up to it ; none of the Dominion specifications called for it. He felt that it would be safe to assume that a wheel load of 7 tons would be sufficient for Colonial and Dominion bridges.

The next problem that arose was the longitudinal distribution of the loading, which Mr Fuller had mentioned. The loading shown on p. 332 was a matter of guess-work. Recently a panel of engineers in the United States had investigated highway loading on American highways. They found that cars were about 4 to 1 in ratio to the lorries, and that the actual loading was very light. They had decided that it could not be adopted for bridge design ; instead, they had suggested the much heavier military convoy type of traffic, a loading which came to 60 lb. per square foot for spans exceeding 1,000 feet. It was interesting to note that at 1,500 feet the American loading corresponded exactly with the new British one ; for greater spans the British loading was even

lighter; the discrepancy lay, therefore, in the shorter-span bridges. There, he thought, it was a matter of passing a judgement on the possible congestion of loading. For spans of 150 to 500 feet, most bridges would be designed to carry two, three, four, or even more lanes of traffic. He thought that, whilst in England it was possible to imagine five 22-ton lorries, eight 10-ton lorries, and a few 5-ton lorries lined up on one lane, it was almost impossible to conceive of another such lane next to it filled with the same quantity of lorries, with the two sets of five 22-ton lorries meeting at the most unfavourable spot on the bridge. If the loading was accepted as a pessimistic but reasonable estimate with regard to one lane, he would ask the Author what he would think about a reduction on the second lane to 75 per cent of the loading, a reduction on the third lane to 50 per cent of the loading, and a reduction to 25 per cent on all other lanes. That would reduce the total loading on wider bridges very considerably and would leave it intact on narrower ones. That reduction could be adopted, he believed, for bridges exceeding, say, 100 feet, and would amount to about 15 per cent of the load for main girders.

He would also recommend the adoption of approximately $\frac{7}{9}$ of the new loading for bridges in the Dominions, the Colonies, and other countries industrially less developed than Great Britain. He based that recommendation on the fact that 7-ton wheels were more likely to be encountered than 9-ton wheels, and that the congestion of loading assumed in industrial England was quite impossible in Colonial or Dominion countries. If a standard percentage such as, say, 75 could be accepted, the loading would approach to within 25 per cent of the American loading.

Like Mr Fuller, he had also calculated the bending moments for longitudinal and transverse slabs 5, 10, and 15 feet long and for longitudinal beams from 10 to 500 feet for the new loading and for the H20—S16—44 loading of the American Standard Specification for Highway Bridges, and he had found that the American loading was about two-thirds of the British throughout the range. The Americans also had transverse slabs designed separately from the longitudinal slabs and from longitudinal beams. Therefore, the scientific derivation of the loading in the two countries was on exactly the same lines. He very much welcomed that addition to the Paper because, as the Author had said, in long-span bridges, an inch of the concrete slab was a considerable reduction of total load. The Americans had a wheel-load of 7 tons and they provided for only one 32-ton truck and not for a 100-ton truck as in Great Britain; therefore, it should be expected that the British loading would be heavier throughout practically the range of spans. He felt that if the reduction for multiple lanes were accepted, there would not be much to quarrel with in regard to the new loading.

He also welcomed very much the introduction in the Paper of special rules for the design of edge beams and curved beams, which had been lacking in the past, and which were quite essential in the design of slabs on bridges

where expansion joints occurred. Presumably the loading could now be used for any type of deck.

In conclusion, he wished to underline the inevitable chancy nature of all live loadings. First, there was a guess as to the maximum vehicle weight; secondly, there was a guess as to how many of them could be on the bridge in one lane; thirdly, there was a guess as to how many lanes there would be at the time. If reasonably pessimistic guesses had been made three times, a loading was obtained which he thought all engineers should consider to be fairly safe for future use. He did not think they should lose any sleep because of fear of overloading or fatigue effects, and he did not think they should waste their time in calculating stresses to two decimal points, as often seen on stress sheets for bridges, when Mr Fuller had just shown that the discrepancy in the loading was something like 100 per cent, although personally he would prefer to call it ± 40 per cent!

Mr A. J. H. Clayton said that he was interested to see how the bridge engineer had to start off with a good deal of engineering judgement in the same way as the general road engineer had to do, in spite of the precision with which the actual calculations could now be made. The bridge, after all, was part of the road, or rather part of the road foundation—a rather specialized and expensive part—and it was not much use having it designed on any basically different principle from the road itself. The road itself was designed on an economic life basis; presumably so should the bridge be, although the economic life of the bridge might have to be greater than that of the road foundation elsewhere.

He, therefore, thought that the Author was right to take into account that the maximum possible loads were not very likely to occur. Presumably some allowance had been made for the infrequency of the normal loading as well as the improbability of the maximum possible load. He had made some rough calculations on that which might be of interest, especially as Mr Kerensky had raised the question; and he was wondering whether those allowances had been made on the basis of any calculations or whether they were a matter of engineering judgement.

There were broadly two sorts of conditions that were normally likely to occur on a bridge. One case was where the bridge was in a completely free-running road. In those circumstances, two vehicles were never likely to be close together one after the other. At 20 miles per hour there would normally be about 60 feet headway when they were running closely together, although they could be somewhat closer. That was rather different from the 25 feet shown in the Paper.

Then there was the case, which was very common, of a bridge that had a road intersection very close to its abutment. In that case there was a queue of traffic regularly every cycle of operations of the control. That queue could extend back over a 100-foot bridge but not much further, except in rare breakdowns or similar circumstances.

Taking Britain as a whole, 0·5 per cent of the vehicles registered exceeded a 5-ton unladen weight. That was probably about equivalent to the 11½-ton load which was taken as the half-loading. Of course, on the road there was great variation. In the case of London roads as a whole, the figure was not much more than 0·8 per cent; but in the industrial and dock areas in London, it could be as much as 3 per cent—which was still quite small. Therefore, on a free-running road the chance of there being on a two-lane 100-foot span bridge two such vehicles passing was quite small (that was, two such half-load vehicles and not the full-load ones, which must be rarer). It would probably occur, with the normal maximum traffic as reckoned by the Ministry of Transport—a figure above that reached on any but a few roads—about 2,000 times a year. That was a fairly big figure, but looked at in terms of alternating stress he gathered, from what he had learned in his younger days, it was not very much.

In the other case, where there was signal control or a level crossing or some other reason for traffic blocks on the bridge, naturally the chance of there being a number of such vehicles on the bridge became greater. The probability of the vehicles standing in one lane on the 100-foot bridge, consisting entirely of vehicles heavier than the assumed half-load, was about 3 in 10,000. It would occur on an average about 65 times a year. The chance of two lanes of a four-lane bridge being so occupied was so small that it might happen only once in 50 years, but could not, of course, be completely ignored.

The figures were intended merely to give an idea of how the problem could be calculated and were not meant to be working figures. They were based on the assumption that the heavy vehicles were distributed in a normal fashion among the general traffic. It was possible for them to bunch to some extent if they were relatively slow-moving. If they bunched, the bunches occurred less frequently. That would increase the probability, which would still be relatively small. There might be special reasons in certain cases for frequent concentrations but that was not normal.

The only other matter to which he wished to refer was the abnormal load. As Mr Fuller had said, the railway engineer had an easier problem. One direction in which he had an easier problem was that he knew that the locomotive engineer was going to get the locomotives up to the full permitted load on the bridges and there practically every train would have a locomotive of the designed load, and occasionally, two locomotives. The road engineer had got to deal with a very widely scattered range of loadings; he was also expected to take those loads which the railways could not take, would not try to take, and did not regard it as economic to make their bridges capable of taking.

That led to the question whether every bridge should be designed for such loadings which were relatively rare compared with traffic as a

whole. There were certain barriers which presumably needed at least one such crossing, such as the River Thames. Most railways had got under-bridges and well as over-bridges; he thought that if the bridge would stand up to the heavy normal load, the abnormal load should be regarded as a very special case.

Mr G. A. Gardner said that for some long time he had been concerned with that unknown quantity, the load on highway bridges, and when he had had occasion to deal with bomb-damage claims, he found that jetties and bridges were classified as land, and he supposed that it would be rational to say that the layman would consider himself entitled to do on the bridge what he could do on the road. Mr Clayton had brought out that aspect of the case.

Things were not getting more easy for the highway bridge engineer who had to deal with greatly increased traffic of a varied nature. There had been a considerable advance in the theory of structures, but the essential problem was not one of design, but one of probability, and it rather astonished him that there was a great absence of statistical data and an ordered probability. The Author touched upon that at a number of places in the Paper.

The Author did not disguise his guess-work. All engineers had to guess, but guess-work was, of course, a very poor substitute for facts. He felt that the Institution should take the matter in hand and try to find out what the traffic was on highway bridges in general. It was not so easy a problem as Mr Clayton's statistics had suggested, because if the bridge was economically designed, it was not desirable that many loads in excess of the design loads should pass over it.

Of course, given a system of wheel loads to cover all comers, the problem was relatively easy; it just rested with the engineers to wrestle with the latest methods of design for slabs and decking. But it was the engineer who did not want that, for he had to keep his eye on the financial factor of safety as well in running his office—and quite properly so. The Author had very ably tried to assist them in that respect, and the fact that very few bridges had fallen down seemed to suggest that a sufficient factor of safety still existed in relationship to current methods, but it would pass the wit of man to know what the actual stress really was in most of the members of a bridge.

A great deal of the consideration of loadings had to be mixed up with the idea, not of what was applicable to Great Britain alone, but of what might be appropriate to other countries, and in consequence the unit-load principle was introduced.

The Author's reference to the provision for loads running near the edges of slabs was very interesting but he was rather surprised to find the Author warning—presumably—engineers that they could not have transverse distribution where the slab was discontinuous laterally. The whole trouble was that the unit loadings were not really the actual loads, but a

load-*cum*-factor for transverse distribution, devised to suit dozens of different types of design of bridge-floor systems.

Mr A. J. Harris said that he wished to call attention to a most important point. It related to the design of a bridge in the transverse direction. Mr Kerensky had said that Great Britain still held the record for size of loading. His own experience, in comparing the Ministry of Transport loading with foreign loadings, was that in Britain the longitudinal loading was a little higher but that no advantage could be taken of any transverse stiffness. The Author's calculations and comments had related mostly to girder bridges with cross-girders but when introducing the Paper, the Author had mentioned the possible application of the reduction of loading in a transverse direction to a monolithic grillage, a type of construction which was becoming more and more common in prestressed concrete where its application, in conjunction with the loading described by the Author, could result in considerable economies.

The second comment he wished to make was very brief and possibly outside the scope of the Paper. The Author had not mentioned horizontal loading caused by traction and braking which occasionally became quite an appreciable factor in such structures as viaducts.

Mr A. L. Somerville wished first to reassure Mr Fuller about the frequency of heavy lorries. He could show to Mr Fuller any day a dozen lorries in a single row each loaded with 15 tons of cement. Mr Fuller had made a point about the length of the lorries. Those were 30 feet long and presumably, even in a traffic jam, they would be at 30-feet centres.

On p. 339 of the Paper, the Author pointed out that main members of large-span bridges which were designed for normal loading could also carry large abnormal loads without further consideration. He then went on to suggest that, therefore, the decks should be similarly designed in order to get a balanced design. A balanced design might have some meaning for the engineer, but he did not think it really had much meaning for the layman who had to pay for it, and before deciding to increase the strength of the deck, an estimate should be made of the cost of such an increase, bearing in mind that the stronger deck was also heavier and would increase the cost of the main members. There might well be sites where large abnormal loads were a very rare occurrence and the total amortization charges of the heavier deck could be compared with the cost of sending such abnormal loads on even a long detour.

Colonel Sidney Green said that he did not agree with the preponderance of speakers who urged a lowering of the loading standard because of the low probability of 22-ton vehicles being present in numbers.

He had noticed, with other types of vehicles, that though the chances of meeting them should in theory be small, concentrations were found in large numbers in areas where their garages were near bridges, particularly in built-up and industrial areas.

He felt, therefore, that the Author's assumptions were on the right lines.

Mr J. E. Jones said that the answer to Mr Fuller had been given by Colonel Green, who had mentioned the possibility of the occurrence of concentrations of heavy vehicles in industrial and city areas, and rather doubted the possibility of that sort of thing happening out on the open road. But if anybody cared to travel a number of roads in the Midlands at night, he would find very heavy concentrations of eight-wheeled vehicles. He granted that they did not follow one another quite as closely as in the Figure in the Paper, but there were numbers of cases where there was no particular reason why they should not and where they might do so if certain things happened. For example, just short of one bridge that he had in mind, there was a transport café ; at about 11 or 12 o'clock at night, one would find ample material to produce the conditions shown in the diagram on p. 332 (*Figs 3*). Therefore, to put it no higher, the Author's guess should at least be given the benefit of the doubt—because it might be right.

He thought there was a case for the admission of a lighter loading for use in certain conditions, although he would qualify that statement by expressing doubt whether there would be very much gained by it in Great Britain. Many of the bridges in the British Isles were of very short span indeed. A few years ago he had taken out some figures which showed that the proportion of bridges of 50 feet or less in total length was probably somewhere in the region of 80 per cent. That meant that the new bridges would quite likely be in the same sort of proportion of the total. He noted that Mr H. C. Adams was present, and no doubt Mr Adams would remember (for he had done it himself years ago) calculating the actual saving that there would be in the total cost of a bridge if the design live load were reduced to as much as half of what was then being taken. If he remembered rightly, the percentage saving had certainly been in single figures, and he doubted whether it had been much more than 5 per cent in all.

It might be said that it was all very well talking about what happened in Britain, but what happened abroad should be borne in mind. To meet such conditions, could not a reasonable scale factor be applied to the basic loading ? He made a plea for some investigation of that point and for some indication from the Author that thoughts were moving in that direction.

Mr W. F. Adams thought that he could claim to be one of the pioneers of the application of probability methods to road engineering, particularly with reference to road traffic. It was because he felt some responsibility that he wished to preach a little caution in the application of probability methods to that question.

Mr Clayton had given some very interesting figures, but he had partly disarmed himself when he mentioned that a probability of 3 in 10,000

represented 65 times a year. It should be remembered that, during the life of a bridge, an enormous number of events had to be dealt with, if the passage of a single load was considered as an event. If one were thinking of designing a bridge on a basis of a load which would cause failure but which, on the average, would arrive, say, once in a hundred years, it should be remembered that the chances of that one load arriving in less than a hundred years were roughly only 40 per cent. (More cogently, the chances of one or more of such loads arriving within a hundred years were about 60 per cent.) For the purposes of such design, he would say that once in a hundred years was once too often.

Mr J. M. Smith thought that a great deal of the probability or improbability could be eliminated with regard to vehicles if there were a maximum limit fixed for the weight of vehicles to be used without supervision on highway bridges. On many occasions when heavy or lengthy loads were to be taken across the roads, it was usual to inform the police.

It would be uneconomical to design every bridge in Great Britain to take a 100-ton load if some motor manufacturer were to decide to build 100-ton lorries. It seemed to him that where the probability was low, it would be far more economical and far more sensible if users of vehicles exceeding, say, 30 tons had to inform the police when they were going over certain roads. There were very few such vehicles, and he thought that would be the economical way of dealing with the matter.

Mr H. W. Kerr asked how the anticipated loads tied up with military needs. That point had occurred to him because in the 1939–1945 war much trouble and diversion had resulted from the restriction on loading of bridges.

Mr W. T. F. Austin observed that several speakers had mentioned the fact that the proposed loading was much heavier than the American loading. For a year he had worked in bridge design offices in America, and he had also noticed that fact.

One point that would not be found in the literature was that there were a number of American engineers who did not like the heavier knife-edge for shear in the loading to which Mr Fuller had referred. There were two different knife-edge loads, one for shear and one for moment, and many engineers, if given a free hand, preferred to use the moment knife-edge, which was the smaller of the two, for both the shear and the moment, because they did not think the higher knife-edge for shear was justified.

Another point about the American loading was that there was a large problem in the United States on roads chiefly, not so much on bridges, from overloading of trucks. Certain loads had been laid down for those trucks, but the operators did not always agree with them and did not observe them; they often overloaded the trucks, with the result that, in certain States, there had been much trouble with the roads being broken up. It might therefore be that the American lighter wheel-loads in the design specifications were not perhaps typical of the sort of wheel-loads which the American truck owners would like to use.

Another thing which had struck him very much in reading the Paper was that, although there had been an advance from the traction engine to the modern Diesel lorry, the load curve had not altered very much. He began to wonder what the next type of vehicle was to be. A question which he had wanted to ask was whether it would not be better to give a train of loads to which bridges should be designed, and if an engineer cared to go to all the trouble to work out the stresses from those loads, he could take advantage of them. If that were done, perhaps the particular wheel-loads would not suit some future type of loading—there might be atomic engines for all he knew—whereas the uniform load and the knife-edge which was suggested might well cover that load.

Mr M. F. Palmer said that, in thinking about the future, they should bear in mind the outlook of the transport contractor. Owing to increases in labour costs, it was more and more economical to run larger lorries, and therefore it was quite probable that in the future the percentage of those lorries would be much greater than it was at present.

With regard to the length of heavy lorries, he had seen lorries of the type mentioned in the Paper built specially for carrying sugar, and he would say that the wheel base was, if anything, shorter than that shown in the Paper.

Mr B. P. Wex asked whether the loading was universally applicable to concrete, timber, steel and, in fact, any form of construction.

Mr S. R. Banks wished to ask a question arising from Mr Chettoe's remark that the British Standard loading was universal whereas the Ministry of Transport Loading was for Britain only. Could the Author explain just what was meant by the word "universal" and how far the loading given in a British Standard would become mandatory in other parts of the world, particularly in the Commonwealth? If it were likely to become mandatory (or perhaps become accepted without much thought), would there be recommendations or suggestions made for scaling it down to correspond with the needs of different countries?

With regard to the spacing of vehicles, would it not be a fact that when vehicles were extremely close together, the impact load would be very small, whilst if they were a little farther apart there might actually be a greater total (live plus impact) load on the span?

*** **Brigadier A. C. Hughes** considered that the proposals for the amended Ministry of Transport loading curve were logically related to the actual loading and their application was simple for all members of the structure. He thought that reductions for spans longer than 75 feet seemed reasonable and more in keeping with the loadings adopted by other countries and particularly by America. Any anomaly of design for short stringers on those long spans was obviated by using the two 9-ton wheels 3 feet apart. The increase in loading for spans of less than 20 feet, based on the two 9-ton

*** This contribution was submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

wheels, resulted in a small reduction for slabs which was justifiable, if only on the obvious grounds that slabs were inherently stronger than beams.

He thought that the suggestion to classify both bridges and loads in terms of units of abnormal design loading was excellent and might well be adopted.

The Author, in reply, said that the answer to Mr Bank's question whether the loading would be universal and mandatory for other parts of the world would depend upon the authorities in the various countries. The British Standards Institution could not compel any country to adopt their specification, but the Author imagined that it would be adopted fairly widely ; there were definite provisions in the specification for scaling down that loading to meet circumstance outside Britain.

Mr Kerensky had mentioned the possibility of reducing the weight of the two 9-ton wheels, 3 feet apart, for bridges in more lightly trafficked countries. It would be perfectly reasonable to scale those down, and to scale down the distributed loading commensurately. In doing so, it should, however, be borne in mind that, for spans of less than 20 feet, those two wheels had been taken with no other load in the span, and their combination had been taken to have an approximately equal effect to other combinations of several, perhaps lighter, wheels. Thus, for instance, in Britain, a 30-ton locomotive might have four axles with spacings of 4 feet, 7 feet 6 inches, and 4 feet, and the heaviest wheel should not exceed 4½ tons, but its effects could be rather more severe than two 9-ton wheels side by side. He thought he ought to give a warning that if those wheels were to be scaled down, due consideration should be given to the possible effects of other wheels in those short spans. There was very little dead load on such short spans, so that more care had to be taken with the amount of live load overstress permitted.

He thought a great deal too much stress had been placed upon the change in the impact factor. There was no doubt that the previous Ministry of Transport Loading had taken into consideration a 50 per cent impact factor. When starting to investigate the new loading, the first step had not been to decide that impact had gone down from 50 per cent to 25 per cent and that the loading could be reduced in consequence. There had been available information that the worst impact condition normally expected in Britain was 25 per cent on one axle or a neighbouring pair of wheels. The effects of modern traffic had then been studied, taking into consideration that 25 per cent impact. If the guess about the number of lorries which could occupy a bridge was correct, apparently the weight of modern traffic had gone up. The problem had not, however, been investigated from that point of view, since it was more satisfactory to relate it directly to modern traffic. He thought it would be extremely unwise to say that because impact had gone down, the loading could be proportionately reduced without considering present-day vehicles.

It was also a mistake to claim that the new proposals were identical

with the former Ministry of Transport loading in the range from 20 feet to 75 feet. That was true only when traffic lanes on bridges designed to carry the new proposals were 10 feet wide. It was rarely that those would be less than 11 feet wide, however, so that there was a consequent reduction of 10 per cent in the loading. For spans in excess of 75 feet, the reduction increased as the span increased, and was quite considerable from, say, 300 feet onwards. In those long-span bridges, the reduction of load in the lanes in excess of two represented a further substantial diminution in total load on the main members.

As to the number of vehicles which had been taken to occupy the length of 75 feet, admittedly those lorries had a rather short wheel-base and admittedly they were packed very closely; but sometimes traffic was brought to a standstill. When bus drivers or drivers of big lorries were stopped in a train, they inched their way up until the bonnet of their vehicle was nearly touching the one in front.

It was unfortunate that the Statutory Instrument did not specify a minimum length for "normal" vehicles, but required that they should not exceed a maximum of 30 feet. An overall length of 25 feet, as had been assumed, was, therefore not unduly short; Mr Palmer's experience, he gathered, confirmed that. It was difficult to say categorically whether three 22-ton lorries would occur in one lane and three in an adjacent lane, but the Author felt it was quite possible; he had seen eight brick lorries, fully loaded (at a guess, he would say overloaded) standing nose to tail—and they might very well have been standing on a bridge.

He did not think he need touch very much on Mr Clayton's statements about the probability of the occurrence of heavy vehicles on bridges. Mr W. F. Adams was more competent than the Author to discuss probability, and he was very pleased to find that Mr Adam's mathematical investigations confirmed the Author's estimates to such a large extent.

As Mr Adams had pointed out, statistics should not be too much relied on in such an investigation. If a combination of loads that was going to cause damage on a bridge occurred once in 50 years, that was a loading which ought to be considered. It would be very difficult to explain to clients afterwards that that was not expected more than once in 50 years!

Mr Fuller had also asked whether the Author was satisfied that his curve was sufficiently accurate for all purposes. He thought that he was so satisfied. The curve had to cover a very wide variety of loading, and he thought the figures quoted by Mr Fuller scarcely gave a right picture. Referring to the Tables (and unfortunately the Tables which had been printed were not so extensive as the Tables which had been produced in the work on the loading), it would be found that in longitudinal girders the maximum deviation from unity was perhaps about 15 per cent less for moment and about 20 per cent greater for shear. That did not seem to him to be very wide of the mark considering the number of uncertain factors and the variety of different combinations of load which

might, in fact, use the bridge. With cross-girders, the effects of the actual normal vehicles ranged from about 65 per cent to 76 per cent of those of the design loading but, in addition, there were abnormal loads of 70 to 80 tons going about the country far more frequently than was generally thought. It so happened that cross-girders with one of those loads running up the middle of them were much more severely affected than by the usual arrangement of loads.

It had been suggested that on small spans an alternative loading consisting of a train of wheels might be provided. That could be done, but again, the question arose of what train or combination of wheels to take. In the practical range of members, it would be seen from Table 2 that the deviation from exact equivalence between the effects of the assumed wheels and the proposed design loading was very small, being, in the main, within $\pm$ 10 per cent which seemed a satisfactorily close approximation. If wheel loads alone were specified, it was still a fact that a single load would be used to represent a variety of combinations, and would, of necessity be an approximation to the worst effects of those, having probably no greater accuracy than the uniformly distributed load. He thought that engineers preferred to design on the basis of a distributed loading when possible.

Mr Fuller had suggested that there should be two different knife-edges, one for moment and one for shear, but Mr Austin had pointed out that the additional knife-edge for shear was not particularly popular in America. If the loading described in the Paper were considered, he thought it was fair to say that the combination of a single knife-edge load and a uniformly distributed load had covered both conditions satisfactorily and certainly within the limits of knowledge of the distribution of loading on the highway and the intensities of the various wheel loads.

Mr Kerensky had mentioned the possible future saving from the change from solid to pneumatic tires. He believed that at present there were in Britain about three abnormal-load-carrying vehicles fitted with pneumatic tires. Until quite recently, pneumatic tires had not been made capable of carrying the loads placed on solid tires. It would take very many years to get rid of all solid-tired vehicles. Hauliers had tied up a lot of capital in them which they would not readily abandon. Eventually, perhaps the loading could be adapted, but at present it was necessary to face the use of solid-tired vehicles and their continued use for a considerable time. The Author knew that when the particular solid tire in question had been obtained, Mr Kerensky had had considerable difficulty in getting a tire of those dimensions and capacities; that had been because most hauliers, instead of using a 16-inch tire, used two 8-inch tires side by side—but the total effect could be, if anything, rather more severe.

Mr Wex had asked whether the loading was universally applicable to concrete, timber, and so on. He thought it could fairly be said that it was universally applicable, but certain points had to be remembered. For

instance, a timber plank deck could not be assumed to be an isotropic slab. That timber plank deck could have one wheel running up the centre of it; obviously slab loading would not be a fitting loading since there would be no transverse strength. That was why a qualification had been put in on beam loading that when members were spaced at less than half the width of the carriageway lane apart, the loading must be taken for a half carriageway lane. Where spacings were less than about 5-foot centres there would occur the worst case for actual loading of one wheel on top of the member under consideration. On the other hand, as the centres were reduced, the distributed load would be reduced, unless some limitation were provided.

When using light-weight slab type decks, it would also be wise to consider the effects of abnormal loads of the order expected to use the bridge, since reductions in dead load effects on slabs might make the overstress from live load serious. Provided proper judgement was used, however, the loading was universally applicable.

There was a possibility of carrying out Mr Kerensky's suggestion of reducing the loading for the second lane to 75 per cent, the third lane to 50 per cent, and the other lanes to 25 per cent. If that were done, he thought it would have to be applied simply to long-span bridges, and without going into the matter very thoroughly, he would not like to give a final statement.

After giving further consideration to that proposal, he had concluded that some further reduction in the load used for lanes in excess of two would be justified. If instead of half the full loading, one-third were used for lanes in excess of two, Mr Kerensky's point would be met without incurring the risk of under-estimating traffic. There seemed good reason to keep to full loading in the first two lanes considered. If that were done, it would also make it possible to apply the relaxation to all loaded lengths, rather than to long-span members only. That was a considerable advantage, since it simplified that statement of regulations and avoided the possibility of the anomalies attendant on changing from one set of rules to another at some arbitrary point.

The point had been made that in large bridges the long-span members could carry a considerable abnormal load and that consequently, as he had suggested in the Paper, it was desirable that the short-span members, should also possess a reasonable proportion of that capacity. It had been suggested that that would be uneconomic. On the other hand, it should be borne in mind that for many miles in each direction a long-span bridge was probably the only crossing of the barrier it negotiated; if abnormal loads were going to cross that barrier, it was almost certain that the bridge would be required to carry them. Consequently, he thought it would always be desirable to make the capacity of all members of long-span bridges sufficient to carry the sort of abnormal load likely to occur in the country in which the bridge was being built. The experience of his

colleagues in the Ministry of Transport who had the frequently difficult job of finding routes for abnormal loads across those barriers would, he thought, confirm his opinion in that respect. He thought the difference in the total cost of the bridge would be much less than might be expected.

He had been pleased to learn from Mr Kerensky that the loading maintained the same proportionate relationship to the American loading fairly consistently. He was not well versed on the subject of American traffic but thought that, generally, the weight of individual units of traffic and the density of heavy traffic in Britain were a good deal greater than in America. In Britain, as had been pointed out by Mr Clayton, there were all sorts of loads which the railways rejected. He believed it was the case that those loads were to a very much greater extent carried on the railways in America, so that there seemed to be justification for British Loading being greater than American Loading.

Both Mr Austin and Mr Palmer referred to the possible increase in the numbers and total weights of vehicles which might take place in the future, whilst the former drew attention to the prevalence of what must be quite gross overloading of vehicles in America. The Author considered that there would always be a desire among hauliers to use heavier vehicles than were at the moment permitted and that, in that respect, bridge design loadings should take into account the possiblity of reasonable future demands. In his opinion, the maximum permitted weights of vehicles and bridge design loadings exerted a mutual pressure on each other. Vehicle designers generally wanted, for economic reasons, to carry heavier loads, whilst bridge designers wished, for exactly the same reasons, to use lighter design loadings. Just as bridge design loadings had to be based on the effects on bridges of actual vehicles, so the maximum permitted weights of vehicles and their axles were to a considerable extent controlled, through Statutory Instruments, by the capacity of bridges. The bridge design loading therefore served two purposes—as a basis for the design of bridges, and as a yardstick against which future developments in vehicle design could be checked, so far as their effects on bridges were concerned. Mr Austin suggested that the adoption of a train of axles—an expedient which at first sight seemd more natural—might not be an advantage, since it might not be suitable for some future type of vehicle. That was true, and, indeed, a train of axles was not necessarily a more realistic type of design loading than that proposed, nor would it necessarily provide a more precise and consequently more economic representation of actual vehicles. Designers could not be asked to work to a large variety of different wheel loads and combinations of wheels, choosing the worst effect for each member considered. It would be necessary to provide a single, simplified train, which would, therefore, be an approximation to several types of loading, and would be subject to the same limitations of accuracy as the present proposal.

Provided the equivalent distributed loading was well chosen, it was

as satisfactory a medium for producing balanced designs for bridges as a train of axles, and a much more simple tool to handle. Equally, it provided a satisfactory yardstick against which proposed developments in vehicle design could be checked, and, if thought necessary and desirable, controlled.

Mr Smith suggested that it would be beneficial to control the movement of very heavy vehicles by law. It was, in fact, necessary to notify Police, Highway and Bridge Authorities of the movement and route to be taken of any vehicle weighing more than approximately 22 tons gross. The Author did not know if any estimate had ever been made of the number of staff or the total cost to the Ministry of Transport, Railway Authorities, County Councils and similar bodies which was incurred in the examination of those notifications and investigations of the capacity of bridges to carry specific loads, but it was sure to be very high. The increase in such costs which would result from adopting a lighter design loading would be considerable and might well exceed the total amortization of the reduced costs of construction. Although it was probable that bridges designed to carry 45 units of abnormal loading would be built only on a limited number of routes in Britain, vehicles weighing 30 to 80 tons or more required to travel to the most unexpected places, and provision had to be made for them, at least on main roads.

The requirements of military traffic, to which Mr Kerr referred, were, the Author considered, not dissimilar to those of the vehicles used in the investigation. Quite a large proportion of the restrictions which prevailed in the years he mentioned still applied, and were the result of the continued existence of so many old bridges which were not designed to carry modern loads. The replacement of those bridges represented a considerable economic problem, but the existence of that problem served as a warning to engineers that design loadings ought not to be pared too finely.

Mr A. J. Harris's comment on redistribution of load in grillage decks was, in the main, answered in the introduction, p. 351. The phenomenon was not confined to prestressed concrete construction, but could equally well be taken into consideration in the design of steel or plain reinforced-concrete bridges. In Germany it had for many years been not only permissible, but compulsory, to investigate and take advantage of those effects. Design in that country since 1945 had produced some remarkable and interesting bridges in which the cross-section had been very largely influenced by the study of grillage behaviour. If advantage was taken of grillage analysis, however, it was essential that that should be based on reliable methods and not on rough and ready approximations, the adoption of which might have serious consequences.

Mr Harris also raised the question of braking and traction forces. Limitations of space prevented anything other than the vertical effects of live load from being dealt with in the Paper, and even in that respect,

only the more important aspects had been covered. The horizontal forces from the abrupt stopping and starting of heavy vehicles could be quite considerable. In his own experience the Author found that the horizontal force from starting an abnormal-load carrying vehicle, especially when it was done on a rising gradient or a rather soft surface, could be very large, and generally greater than the corresponding braking force. Where abnormal load carrying vehicles were concerned, either of those forces could be applied on a very short length—being concentrated at the two driving axles of the towing locomotive when starting, or on the two axles of one bogey of the trailer when stopping. Provision for those forces was being made in the regulations. They amounted to 1 ton per unit of abnormal loading, when that was used, and ranged from 10 to 25 tons total for normal loading, depending on the span.

The closing date for Correspondence on the foregoing Paper was the 15th May, 1954, and no contribution received later than that date will be printed in the Proceedings.—Sec. I.C.E.

Paper No. 5944

"Renewal of the Steelwork of Railway Bridge over Odzi River, Southern Rhodesia"

by

Kenneth John Pitkin, A.M.I.C.E.

(Ordered by the Council to be published in abstract form.)†

Introduction

Site.—The single-track railway line from Beira in Portuguese East Africa *via* Umtali to Salisbury, the capital of Southern Rhodesia, is one of the two most important routes from the sea to the Colony. This line crosses the Odzi River 19 miles west of Umtali, and 222 miles from Beira. Like the majority of Central African rivers, the Odzi is very low during the dry season, roughly April to October, but rises many feet during the rainy season.

Historical.—The river was first bridged in 1899 when the line was constructed from Beira to Salisbury by the former Beira and Mashonaland Railway Company. The bridge consisted of two 100-foot spans and was designed to carry a maximum locomotive axle-load of 9 tons 12 cwt.

During a severe flood in 1918 the masonry pier in the centre of the river was swept away and the bridge collapsed just behind a train. A small ferry service was instituted, and passengers and light goods traffic transhipped across the river, while a temporary timber bridge was erected.

On a new site upstream of the old bridge, the present bridge—the steelwork of which has just been replaced—was erected. It consisted of four spans with three piers, two of which were founded on steel cylinders sunk through about 15 feet of moving sand to rock, and the third founded on rock with normal foundations.

From second-hand material the bridge was rebuilt of three double Warren-type deck spans, 91 feet 2 inches between centres of bearings, and of one plate-girder deck span 73 feet $3\frac{1}{2}$ inches between centres of bearings.

This bridge was completed and in service by 1920. In 1930 it was strengthened. With constantly increasing traffic and axle loads it became necessary after the 1939–45 war to renew the superstructure completely, but the concrete abutments and piers were sound.

Character of work.—The steelwork had to be renewed with the minimum interruption to traffic over the single-track bridge; on account of the

† The full MS. and illustrations may be seen in the Institution Library.—Sec. I.C.E.

nature of the river-bed, any temporary bridge would have been too costly and staging in the river was considered unwise, owing to a possible sudden rise of water from floods. A lift of rail-level of about 3 feet to improve approach grades was considered desirable whilst renewing.

The consulting engineers to the Rhodesia Railways, Messrs Freeman, Fox and Partners, were asked to design a new superstructure to carry 20 units of British Standard loading, and to evolve an erection scheme to meet their requirements.

Technical Description of Work

The existing bridge comprised four spans carried on three river piers about 30 feet high, and on two dissimilar box-type abutments. The Umtali end span was a simple open-deck type plate girder; the other three spans were lattice girders with superimposed cross-girders and stringers. Height from rail level to top of piers was 11 feet 6 inches, to the underside of lattice girder spans 11 feet 3 inches, and to highest recorded flood level (1925), 11 feet.

Design of new structure.—The new bridge was designed as a riveted plate girder with a uniform depth of 7 feet over main angles, with sleepers resting directly on girder flanges, provision being made for launching over rollers. The two centre spans are continuous, with short cantilever extensions beyond piers Nos 1 and 3, and the end spans are simply supported and linked to the cantilever extensions by expansion bearings. Maximum depth from rail level to underside girder is 8 feet 6 inches. Pier No. 3 (being founded on rock) carries the fixed bearing of the continuous portion of the bridge.

The bridge and most of the major erection steelwork was fabricated to the consulting engineers' design by the Cleveland Bridge and Engineering Company, Limited, of Darlington, England. The total length of the bridge is 348 feet 7 inches and its weight is 168 tons; the weight of erection steelwork was 45 tons. It was shipped in sections; the maximum weight of these was 6 tons 17 cwt. (*Fig. 1.*)

Outline of erection scheme.—The old bridge, together with the approach banks, was first raised 3 feet 3 inches and supported on temporary grillages on the pier tops. In the resultant space, transverse traversing beams were placed at pier-top level, with projections upstream and downstream supported by inclined struts springing from the pier bases. The new bridge was assembled section by section parallel to, and upstream of, the old one, upon launching rollers on a stage built part-way out from Salisbury abutment, and launched over rollers placed over traversing beam extensions. With the new structure in position alongside the old, the rollers were removed, and the weight of both bridges transferred on to ball-carriages running on the traversing beams on the piers and on corresponding tracks at the abutments. The two structures were then rolled bodily and simultaneously downstream, the new one coming into line-of-way and the old

one out on to the downstream beam-extensions, where it could be dismantled. The whole traversing operation and subsequent transference of weight of the new bridge on to its bearings, was planned for execution in a single operation of 12 hours.

The erection gear, supplied from overseas, comprised traversing beams and struts, rollers, launching-nose, draw-beam, ball carriages, hydraulic capsules for weighing reaction at expansion bearings at pier Nos 1 and 2, and an unloading and assembling gantry, which was not used. All other items were designed and supplied by the bridge department.

Fig. 1

NEW STEELWORK UNDER ERECTION

Detailed procedure.—The new bridge was erected 13 feet 6 inches upstream of the old bridge, the old bridge first being lifted 3 feet 3 inches in three stages; the first two lifts under 8 hours' occupation, were of 1 foot 3 inches and 9 inches respectively. Grillages, supplied departmentally, were used to support the old spans and were so designed that the new steelwork could be placed around them.

The Umtali end span was shallower than the existing double Warren, and it was tied to the end double Warren to lift with it for jacking. To raise these lattice spans, jacking brackets were made by burning out end plates to the shape of 12-inch-by-8-inch R.S.J.'s, the joists being pushed

through and cleated. Duff-Norton self-lowering screw jacks of 50- and 35-short-ton capacity were used.

Assembling of new steelwork.—The girders of the new bridge were set up on trolleys by 6-ton crawler cranes, and all bracings etc. were bolted on; the section was then brought forward on to rollers on the launching stage; the next section followed in the same way and was bolted to it and this sequence was continued. Girder splices and all connexions were riveted on the launching stage; several sections, joined together, gave sufficient counterweight to prevent overbalancing before the forward end reached the rollers on pier No. 1.

Double rollers were used on all piers for launching, with single and double rollers on the launching stage. These rollers had side guide-rollers to keep the bridge in position.

Grillages.—To make up the difference in height, grillages were designed and fabricated with the bridge. First, two layers were placed on pier tops, then the top three layers were erected on the outriggers under the new spans, and bolted to the new bearings and traversed with the spans. They thus came in over the other two layers so that they could be bolted to them. Extreme accuracy in setting out was essential.

Piers.—On completion of the first two lifts of the old bridge, holes for holding-down bolts for piers Nos 1 and 2 were drilled. Traversing beams were placed and levelled and outriggers placed from tops of pier cylinders.

Work on pier No. 3 was complicated because a concrete stool had to be demolished and a steel trestle substituted. Traversing beams were placed and levelled as on the other piers and outriggers were placed in position. These strut beams were placed at ends of cantilever on a special foundation carried down to rock since there were no cylinders as on piers Nos 1 and 2.

Abutments.—At the abutments, the cantilever portions of traversing beams were carried on timber supports.

Launching of New Bridge

A 5-ton hand winch, with a 3 and 2 tackle for the final pull (a 2 and 1 for earlier pulls) was used and was adequate for the purpose.

The first section of bridge had the launching-nose and draw-beam bolted to it before being pushed on to launching stage rollers; a cable from the winch was attached to the draw-beam. Launching was carried out on the principle of a move forward and the attachment of a further section to the tail. The special articulated joints between the second and the third sections had to be locked during launching.

The launching of the new bridge across the openings proceeded smoothly. Maximum labour force was 9 Europeans and 40 Africans. The first section was placed on the rollers of the launching stage at the beginning of August 1950, and the new bridge was finally across on the 5th October, 1950.

Launching rollers were removed and bearings placed under new spans.

Expansion bearings were strapped to prevent movement. With all sleepers and rails placed in position, the new bridge was ready for traversing, and a 13-hour occupation of the line was arranged for the 1st November, 1950, commencing about 05·00 hours.

Traversing Old Bridge out and New Bridge in

Traversing carriages were placed under the bottom layers of the new grillages on piers Nos 1, 2, and 3. The carriages were made to ride on 2½-inch diameter steel balls, which in turn ran on top of the traversing beams. On the abutment the launching rollers were used, being placed upside down and traversed.

Traversing was done by ratchet jacks, two on each pier and one on each abutment, working against jacking brackets bolted to traversing beams.

When the new bridge was jacked up, the carriages were removed, and the span lowered into place on layers of grillages already in position. The bridge fitted well for line and level and the first train crossed it at 16.50 hours, the job being completed in 11 hours.

Dismantling old bridge.—The old double Warren spans were joined together over the pier outriggers, the pieces burnt off and dropped into the river, and later pulled out and sold for scrap. During this operation the grillages under the new bridge were concreted in.

The Paper is accompanied by thirty photographs, and eight sheets of drawings.

CORRESPONDENCE

on a Paper published in Proceedings, Part II, February 1954

Maritime Paper No. 24

"Overhaul and Repair of Lock Gates in the Port of London" †

by

John Thomas Williams, B.Sc.(Eng.), A.M.I.C.E.

Correspondence

Mr D. E. Glover observed, with regard to a system of maintenance, that dock gates appeared to be a difficult subject for the medium-term maintenance-contract system. The Author had hinted at that when stating that difficulty had been experienced in making as much progress as desired during the early stages of the work.

Delay in itself was serious enough, but might eventually be overcome by the establishment of a priced schedule. A far greater fault might exist in the establishment of a schedule based upon existing practice, which might discourage attempts to establish more economic methods of maintenance. For example, Mr Glover knew of the development of a process whereby strips of greenheart were fixed to the trimmed face of worn heel-posts, with considerable economy in labour and material, but it was at least questionable whether that departure could have been made in the time available if discussion with contractors as to price had intervened, or even if it would have been profitable in those circumstances.

Dock-gate repair was very specialized work and the establishment of a body of craftsmen with long experience of it was of great value. Such men not only brought skilled craftsmanship to each job, but, through continual work on gates, amassed a wealth of detailed knowledge which might prove invaluable in cases of emergency. It was difficult to see how that state could be reached unless the work was carried out continuously by the same organization. Where the authority was large enough to keep an adequate maintenance staff fully employed, direct labour would appear to be a technically desirable system, particularly where slipping or dry-dock facilities were available within the organization.

Information regarding the average length of life of gates before major

† Proc. Instn Civ. Engrs, Part II, vol. 3, p. 44 (Feb. 1954).

overhaul in situations where the majority of shipping passed on a "level," led Mr Glover to the conclusion that that was in the region of 25–35 years, or more than 10 years in excess of the figure given in the Paper. His figures were based upon information from sources where the total life of gates had averaged 50 years, but that figure should be used with caution, for it included timber, iron, and steel gates. It would be interesting to know if any steel gates stepped in Great Britain had yet reached the end of their life. Perhaps the accelerated deterioration in London was accounted for by the large number of movements necessary where dock water was maintained above natural high water level.

To those who had been used to the abandonment of rollers, the proposal to provide them on new gates would come as a surprise. Rollers necessitated complications in structure which added appreciably to maintenance costs. The choice appeared to be between larger anchorage forces and greater frictional wear at the lower extremities of the heel-post on the one hand and almost certain difficulties in operation where the roller path was liable to accumulate silt and such obstructions on the other. Where that happened total stoppage might occur, but from the structural point of view the most serious condition was, perhaps, attained where the obstruction was not sufficient to prevent operation, but great enough to throw heavy stresses on to the gates themselves. In Humber ports in recent years, rollers had almost been eliminated in various types of steel gate up to weights of 240 tons per leaf.

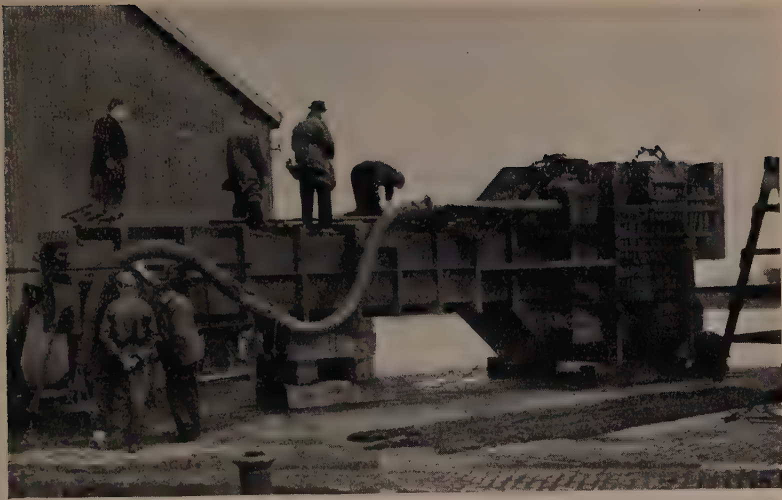
Would the Author say whether that possibility had been considered by the Port of London Authority and, if it had been, why it was discarded? Perhaps the frequent gate movements referred to earlier were sufficient to keep the paths free from serious obstruction.

Mr Glover observed that the figures for costs in the Paper were interesting but did not appear suitable for comparison with figures from other ports. They showed a wide range in themselves, which was to be expected, for it was unusual to find identical repairs required by gates of similar age and construction. No mention was made of inter-departmental charges which varied between ports and which might have considerable effect on final costs.

He suggested that a more suitable form would be man-hours per unit of work; Table 4 had been compiled from recent work at Hull to show that point.

The Author's belief that the cost of repairs in London was twice as high as in the provinces, was difficult to understand. Shipbuilding wages were based upon a national uniform plain-time rate. Repair work was generally paid on those rates plus 3*s*. per week, with exceptions in the Thames and Bristol Channel areas where agreements made prior to the national rate still operated. Mr Glover had been unable to obtain details of those but it might be said that, in shipping circles, London repairs were generally regarded as being between 10 per cent and 15 per cent greater

Fig. 31



General View of Caisson laid on its Side

Fig. 32

Caisson Chamber—Outside. Canvas being fixed over Rope Packing, Note Ballast round Sides

Fig. 33

Caisson Chamber—Inside, showing Pipes for De-watering. Also "Sausage" of Old Rope Packing

Fig. 34

Granite Sill, showing Repaired Joint

than the average for other United Kingdom ports. Perhaps further light could be thrown on the Author's conclusions by an examination of overhead and supervisory costs.

TABLE 4.—LABOUR ON DOCK-GATE REPAIRS

(Average figures based upon complete overhauls on slipway)

Item	Unit	Grade	Man hours
Clean and paint interior, three coats .	sq. yd	Painter	4
Clean and tar exterior, three coats .	sq. yd	Labourer	5·5
Renew condemned plates (riveted) .	sq. yd	Riveter	24
		Plater	16
		Holder-up	20
		Plater's helper	20
		Lad	16
Renew greenheart heel-post . .	cu. ft	Carpenter	66
		Labourer	8
Renew greenheart sill and mitre .	cu. ft	Carpenter	45
		Labourer	6
Renew fendering	cu. ft	Carpenter	4 to 7
		Labourer	2

Mr J. R. Witty commented upon the information given regarding the methods adopted for repairing underwater masonry; the ingenious method adopted at Regent's Canal Dock and described by Mr Tripp, had been of great interest. At Hull recently there had been a similar problem at Albert Dock. The joints of the masonry sill at the outer gates had become defective and repairs had been necessary to avoid loss of dock water, which had been reducing the availability of the dock for shipping. Opportunity had therefore been taken to carry out the repairs to the masonry sill while the gates were removed for overhaul. In that case, however, a tunnel chamber, such as that used at Regent's Canal Dock, could not be used, for the draft of vessels using the entrance was too great to allow a reduction of the depth of water available.

After very careful consideration, it was decided to construct a welded-steel chamber, 7 feet square, in which masons could work on the stonework on the entrance bottom in the dry. Access to the chamber was by means of a 3-foot-square shaft, also of welded steel, fitted with a steel ladder and fixed to the chamber. The whole unit was about 26 feet long and was fabricated at the Docks and Inland Waterways Dockyard, at Hull. (*Fig. 31* gave a general view of the caisson.) The dead-weight of the caisson was 8·7 tons, and approximately 11 tons of ballast, consisting of old rails, had been placed round the chamber, the total weight as placed on the dock bottom being about 20 tons. The depth of water round the caisson, when in position, ranged from 5 feet 6 inches to 17 feet.

A seal had had to be made between the underside of the chamber and the surface of the masonry sill. That had been effected by fixing timbers underneath the chamber, to which flexible pads or "sausages," consisting of canvas fitted round old-rope packing, had been attached (see *Figs 32* and *33*). Under the weight of the caisson, the packing had fitted closely to the shape of the sill. Some difficulty had been experienced with mud which collected rapidly on the sill, and the assistance of a grab dredger and a diver had had to be enlisted fairly frequently to remove the mud. The final adjustment to the placing of the caisson had been possible only by the diver blowing away mud by water pressure through flexible hose from the dock hydraulic-pressure mains.

The mode of operation of the caisson had been governed by the fact that shipping could not be interfered with at high water. The caisson had been carried therefore, on a lifting barge, from inside the dock to the entrance sill, very shortly after high water. It had then been lifted by the crane attachment on the barge, and placed on the entrance bottom over the sill. *Fig. 33* showed the 6-inch pipes and the 4-inch (auxiliary) pipes which had been fitted in the caisson so that the water inside could be pumped out. It had also been found necessary to fit ejectors to de-water the chamber for the last few inches. When the chamber had been emptied, a mason and his mate had been able to descend, and carry out very satisfactorily the repairs to the granite stones. A number of old stones had been removed and replaced by new pieces of granite dressed on the quay close by. Other joints with only small defects had been grouted up with concrete. Generally with one placing of the caisson, two to three joints could be dealt with in the period available, although two placings at a particular group of joints had sometimes been necessary to lift out the old masonry, prepare a new bed, and fix new stones. (*Fig. 34* showed a repaired joint.)

About 4 to 6 hours' working time in the chamber had been possible, depending upon the time taken to place the caisson before the latter had to be filled with water, lifted off the sill, and taken away by the barge to a conveniently close site inside the dock. Since the shaft was open to the atmosphere, the air breathed had been practically normal, although warmer. When pneumatic drilling was in progress, the atmosphere had, to some extent, been dust laden, and alternative methods had been introduced to assist the ventilation. A canvas tube, 9 inches diameter, with an enlarged top, after the manner of a ship's funnel, had been let down the shaft, until the bottom end had been at or near the level of the man working in the chamber. The top end, which had been above the top surface of the shaft, received the wind and deflected it down below. A pipe had also been introduced to deliver air under pressure, but that had tended to cause too much draught.

The men concerned experienced no serious discomfort, and the repairs to the stones had been very efficiently carried out. Mr Witty had himself descended a number of times into the chamber and had been impressed by

the satisfactory working condition. In all, thirty-seven joints had been repaired, twenty-four requiring new pieces of stone. To do that, the operation of placing and removing the caisson had been performed seventy-two times.

An average time for the operation of placing, that was, bringing the caisson to the site, placing it in the required spot, and making a watertight seal, was about 2 hours. The actual de-watering took approximately ½ to 1 hour, the 6-inch pump removing the body of the water down to a few inches off the bottom sill, and the two ejectors completing the operation. The latter had been kept going, so that minor leaks in the chamber had been satisfactorily dealt with.

During the work, the old disused roller paths of the gates had been encountered and the caisson, when placed over the paths, which were about 2 inches deep, had to be fitted with deeper " sausages " to take up the shape of the path and make a seal; that proved quite satisfactory. No difficulty in making the caisson watertight had been experienced with grooves worn by the gate chains, no doubt owing to the hard nature of the granite. A number of the masonry joints had been much worse. Minimum trouble had been experienced in sealing the caisson when the edges had been positioned away from a joint, and that had been done wherever possible.

The repairs to the sill had been started on the 24th August as soon as the gates had been removed, and finished on the 5th November, 1953, a matter of 11 weeks.

The unit was a useful item of plant for future use. Arising from the experience gained in the first operation, it was proposed to fit two additional ejectors and also fixed equipment on the caisson for blowing mud away while it was being placed in position.

It should be noted that no repairs were required to sill joints close to the gate recesses; that information had been obtained from previous examination by diver. The caisson could be modified for that purpose by making one side of the chamber removable, so that it would fit up against the wall.

The Author, in reply, agreed to some extent with Mr Glover that dock gates were a difficult subject for a maintenance-contract system. The difficulties that had occurred had arisen from the lack of precise information about the condition of gates before being lifted out. The economic application of a priced-schedule system depended largely on reasonably accurate information about the condition of the gates before work was put in hand. Where extensive damage or deterioration was suspected, it might be difficult to apply a priced schedule because it was not always possible to survey the gates properly under those conditions.

What Mr Glover had said about direct labour was very much to the point, but in view of the many trades employed on a lock gate, a carefully

integrated programme of overhauls would be necessary to ensure the full employment of an economically balanced labour force.

The Author had no personal knowledge of any steel gates in Great Britain which had reached the end of their life by deterioration alone.

With regard to rollers on gates in the Port of London Authority, there had not been any undue difficulties, so far as the Author was aware, in operation because of silt or other causes. The new gates which had been fitted with rollers were required to be interchangeable with existing gates and to fit the existing anchorages. Where new gates were being fitted in new construction, or extensive reconstruction, it was probable that rollers would not be used.

The costs tabulated in the Paper had been included to give some idea of the amount of work which it had been found necessary to do on the gates concerned and also to show that, under the circumstances which had obtained, the decision to repair some of the gates instead of ordering replacements had been justified.

Mr Glover's Table of man-hours was very interesting but other trades were involved and it was there that custom and practice, particularly in a port like London, had a bearing on the costs. The Author felt some disappointment that that point had not received more comment.

The Author had read with interest Mr Witty's description of the caisson which had been used at the Albert Dock, Hull. The photographs showed very clearly the construction of that caisson which should be of great interest to all engineers who might be faced with a similar problem. The effectiveness of the seal when placed over the roller paths was worthy of note, and there appeared to be no reason why that type of caisson, by suitable modification of the lower part, could not be used for most types of sill.

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